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Dome of the Wisconsin Capitol

SYNOPSIS—In many large public buildings a lofty dome is one of the prominent architectural features, and the structural design of the steel framework of such domes comes within the province of the engineer. This work presents many special problems, both in general design and in the details of structural members and connections. The accompanying article describes in some detail the dome of the new Wisconsin State Capitol at Madison, Wis. It includes also the steel tower which forms the support or substructure of the dome, rising from the foundations and carrying the dome independently of the main portion of the building.

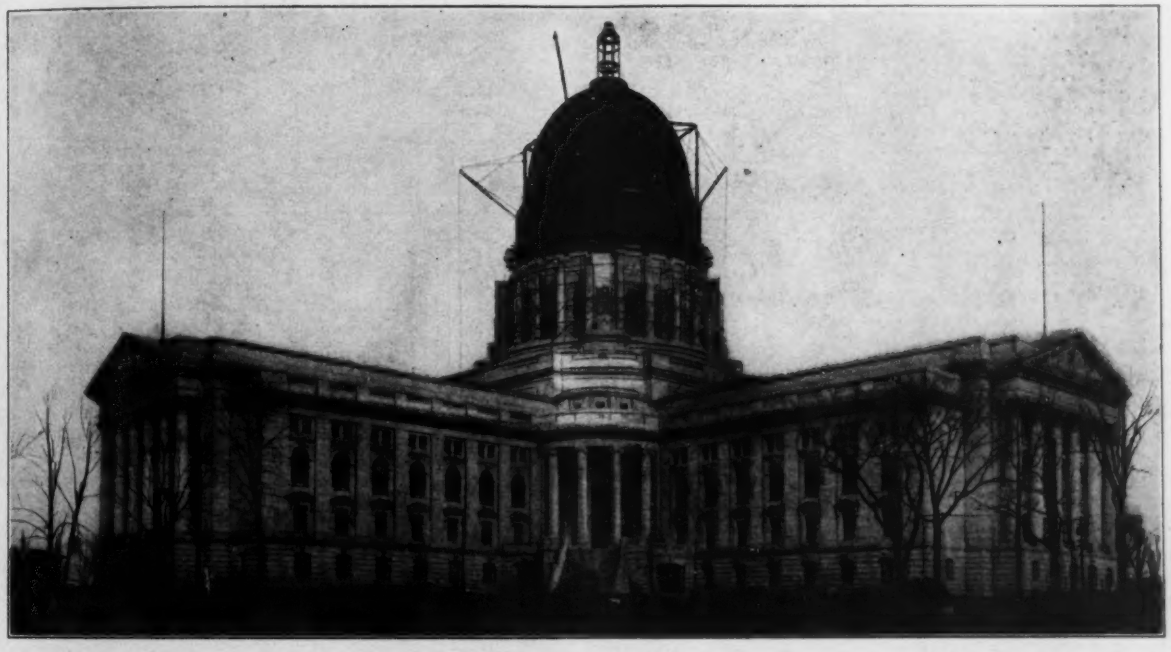

FIG. 1. THE NEW WISCONSIN STATE CAPITOL AT MADISON, WIS.
(Geo. B. Post & Sons, New York, Architects.)

The Wisconsin State Capitol, at Madison, Wis., occupies a commanding position in a 14-acre park on the summit of an elevation in the center of the city, and the new structure which is now approaching completion is not only handsome in its architectural design, but presents features of particular interest in the structural design of its great steel-framed dome.

The history of the Wisconsin State Capitol may be summarized as follows, from information furnished us by Mr. Thwaites, of the State Historical Society of Wisconsin. The first capitol of Wisconsin Territory was erected in 1836 at the village now called Leslie, Lafayette County. In the winter of 1836-37, Madison, at that time uninhabitated, was chosen as the capitol, and work on the new territorial capitol at that place was begun in the spring of 1837. This capitol cost $60,000 and was some three or four years in building. In 1857 work was begun on a new State capitol (the one now in course of demolition) and this was finished in 1869. In 1882 the State legislature appropriated $200,000 for the construction of two transverse wings. On Feb. 27, 1904, a large part of the interior of the capitol (with its contents) was destroyed by fire. In 1905, the legislature passed an act authorizing the construction of a much larger building on the same site, and limiting its cost to $6,000,000. The work on the new capitol, now in course of construction, was begun in 1907. Three of the four wings and a considerable portion of the dome have now been virtually completed. The fourth and last wing will probably be commenced during the present year. The work is being paid for as it progresses, the money being provided from the surplus funds of the State treasury. This somewhat restricts the rate of progress, while another restriction is the necessity of providing accommodation within the building for the government

departments and for the biennial meetings of the legislature.

The construction of the new building was commenced in 1907, and is expected to be completed by 1917. This building is cruciform in plan, 434 ft. long over the arms, with a segmental pavilion in each corner; in these pavilions the main entrances are placed. The ends of the arms or wings (63 ft. wide) face the four points of the compass, while the main entrances front upon four main avenues radiating from the park. From the center rises a circular tower surmounted by a great dome 90 ft. in diameter at the base, and with its summit 236 ft. above the datum level, which is that of the ground floor. Above the dome rises the lantern, to an elevation of 267 ft., and the top of this forms the base for a statue 16 ft. high. The architects for the building are Geo. B. Post & Sons, of New York, to whom we are indebted for information and drawings. Fig. 1 is a general view of the building, showing two of the new wings and the completed frame-

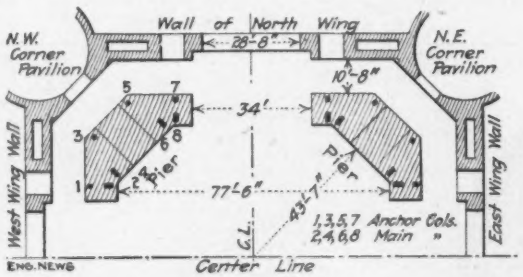

FIG. 2. HALF PLAN OF STEEL TOWER AND MASONRY PIERS SUPPORTING THE DOME OF THE WISCONSIN STATE CAPITOL

work of the dome, with the masonry completed to the circular colonnade around its base.

GENERAL DESIGN OF DOME

The dome and its supporting tower form a steel structure independent of that of the wings and extending to the foundation footings, but the design of the tower or dome-supporting substructure is a combination of steel-skeleton and bearing-wall construction. No difficult problems were involved in the foundations, as the substratum is of a very favorable character and the footings rest upon a formation of a hard-packed mass of gravel having stones up to 3 in. in size. The use of a tower or substructure of steel-skeleton construction to support the dome was decided upon by the architects, after careful computations, as being more economical than solid masonry construction, and also as permitting a more exact analysis of the stresses on the structure. The design of the steel framing of this great dome and its supporting structure was worked out by the engineering staff of the architects.

In brief, four masses of reinforced concrete form the footings for eight concrete piers in which are embedded the steel columns of the substructure. These columns extend to Elev. 78, and are capped by plate girders forming a ring-course octagonal in plan (with alternate long and short sides). Across this ring course are placed cantilever girders, in radial position. The inner ends of the cantilevers carry columns rising to Elev. 174, from the tops of which columns spring the arch-trusses of the dome. Above this is the lantern. The outer ends of the cantilevers carry columns supporting a stone colonnade around the barrel or cylindrical base of the dome.

The superstructure, from the supporting platform on the ring course at the top of the tower (at Elev. 78) to the top of the lantern, was designed as a steel-skeleton frame carrying the total weight to the platform at the ring course. The 24 pairs of columns supporting the dome ribs carry a total load of 11,000 tons, or about 458

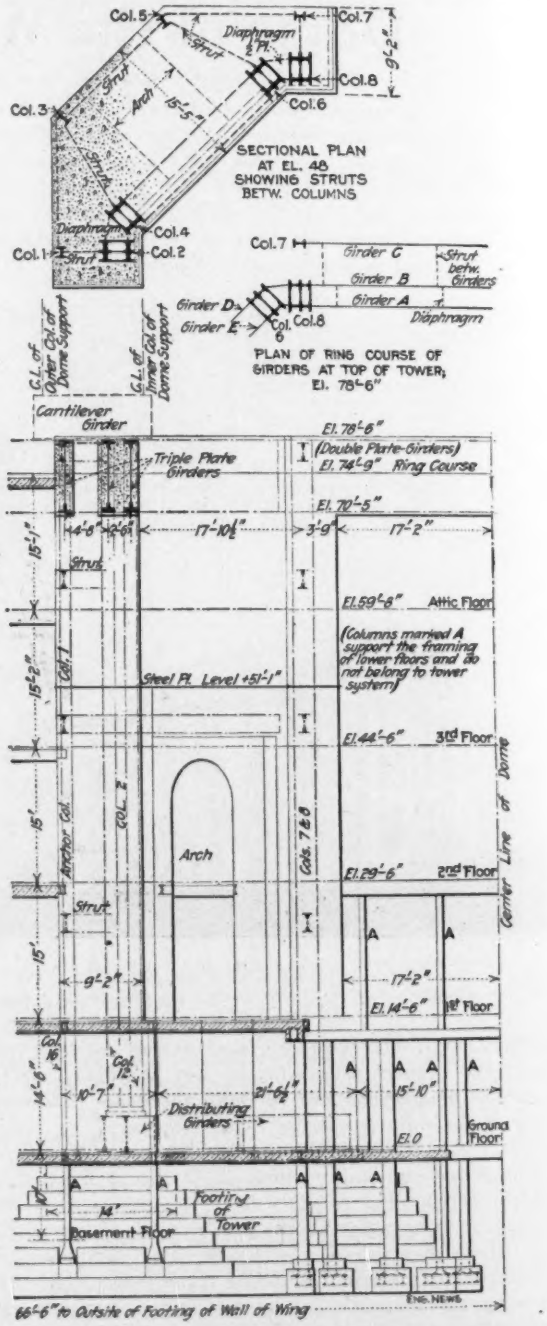

FIG. 3. SECTIONAL ELEVATION AND PLAN OF ONE OF THE PIERS, WITH THE FOOTINGS AND THE PLATE-GIRDER RING-COURSE WHICH CARRIES THE DOME

tons per pair, which is the load carried by each cantilever girder. This platform load of 11,000 tons is supported by the eight pair of main columns of the substructures. A load of about 1700 tons at the base of each pair gives a pressure of about 15 tons per sq.ft. between the grillages and the tops of the footings, and a load of 1300 tons from each of the eight concrete piers (Elev. 0 to 78) produces a pressure of 5 tons per sq.ft. on the top of the footings. The total load from the whole substructure and dome structure, amounting to 30,000 tons, is carried by four reinforced-concrete foundation piers on footings about 16 ft. thick, and produces a soil pressure of about 3½ tons per sq.ft. at the bottom of the footings. The eight piers are of stone concrete (1:2:4), and each contains about 750 cu.yd. The four reinforced-concrete foundations or footings upon which the piers rest contain about 780 cu.yd. each. This makes a total of about 9000 cu.yd. The structural steel amounts to about 3000 tons; 900 tons of this is in the substructure (to Elev. 78) and 1400 tons in the superstructure and dome.

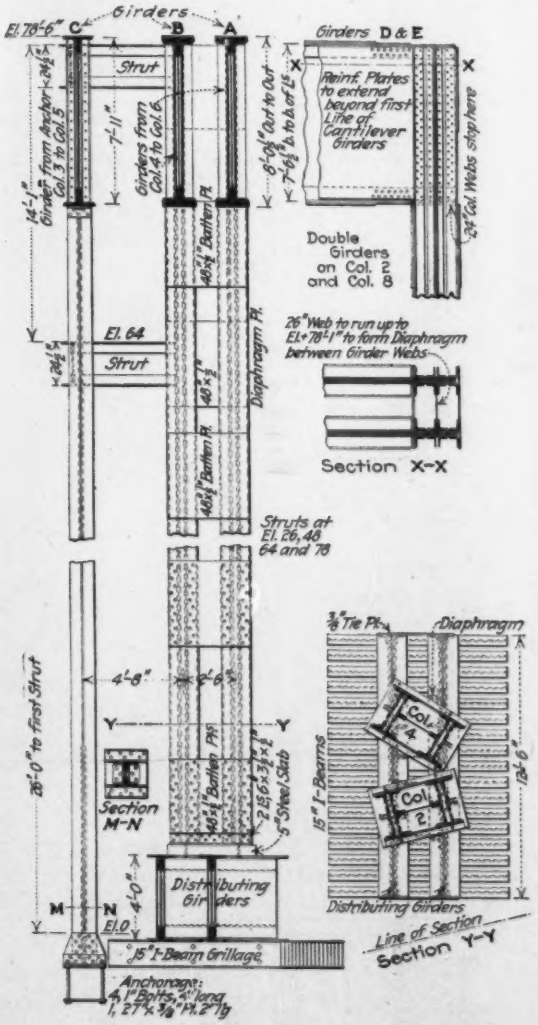

FIG. 4. DETAILS OF COLUMN OF TOWER, SHOWING THE DISTRIBUTING GIRDERS AND GRILLAGE AT THE BASE AND THE PLATE-GIRDER RING-COURSE AT THE TOP

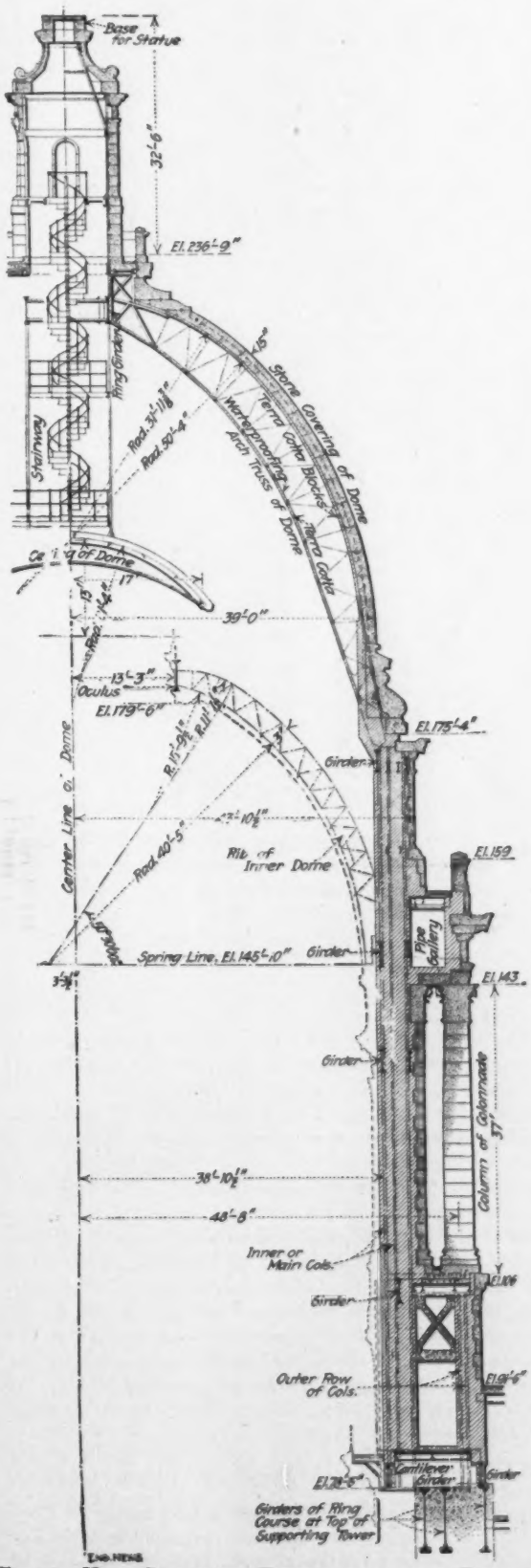

FIG. 5. HALF-SECTIONAL ELEVATION OF THE DOME OF THE WISCONSIN STATE CAPITOL, MADISON, WIS.

The Supporting Tower

The supporting tower or substructure, which extends from the top of the footing to Elev. 78, is designed to carry the entire weight of the superstructure (consisting of the barrel and the dome proper). It consists of 16 main columns arranged in eight pairs at the corners of an irregular octagon having sides about 42 ft. and 22 ft. long, between the columns. Behind each main column is an anchorage column, connected to the former by plate-girder struts. The 32 columns are inclosed in eight massive concrete piers, which are connected in pairs by walls pierced with archways. The arrangement of the columns and piers is shown by the plan, Fig. 2, while Fig. 3 gives a plan and elevation of two of the connected piers, with their footings and their capping or ring course of heavy plate girders. In this plan, the inner columns, Nos. 2, 4, 6 and 8, are the main bearing columns, while the outer columns, Nos. 1, 3, 5 and 7, are the anchor columns for the cantilevers carrying the dome proper, as described below. These great piers serve both as bearing walls and as anchorages for the anchor columns, and

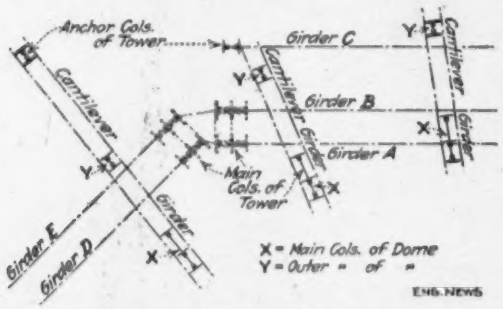

Fig. 6. Plan Showing the Arrangement of the Radial Cantilever Girders Which Rest across the Girders at the Top of the Tower and Carry Two Rows of Columns for the Dome

they serve also to bring the loads to the center of gravity of the footings.

The main columns are connected at different levels by horizontal octagonal rings of plate girders, which serve to carry the floor framing of the different stories. This framing is mainly of 12-in. I-beams, with H-beam supporting columns in the interior space. The columns carry a ring course of 8 ft. plate girders forming a octagonal platform for the superstructure. The long sides of the octagon, which are at right angles to the center lines of the four wings of the building, consist of three girders each; two of these rest on the main columns and the third rests on the anchorage columns. The short sides of the octagon, which are at right angles to the center lines of the four corner pavilions, consist of two girders each; these are carried by the main columns, the tops of the anchorage columns being connected only by struts in these short sides. This is shown by the diagram plans at the top of Fig. 2.

Structural Features of Tower

In the substructure of the dome (extending to Elev. 78 ft. 6 in.) each of the 16 main columns is of H-section, with double (separated) webs 24x3/8 in. for each of the two sides and a single central web 26x1/2 in. There are 16 vertical angles 6x4 in., and at each of the four edges of the H is a cover plate 16x3/8. Across the open sides are batten plates 48x1/2 in. riveted through the cover plates and corner angles. This column construction is shown in Fig. 4. The base of the column has no shoe, but simply a pair of horizontal angles across the bottom batten plates, the vertical plates and angles being faced to rest on a 5-in. steel slab 50x34 in., which is faced on top and bottom. This slab rests across a pair of 48-in. distributing girders 12½ ft. long (with webs 30 in. apart), and these in turn rest on a grillage of 15-in. 70-lb. I-beams 8½ ft. long, spaced 9-in. centers. The

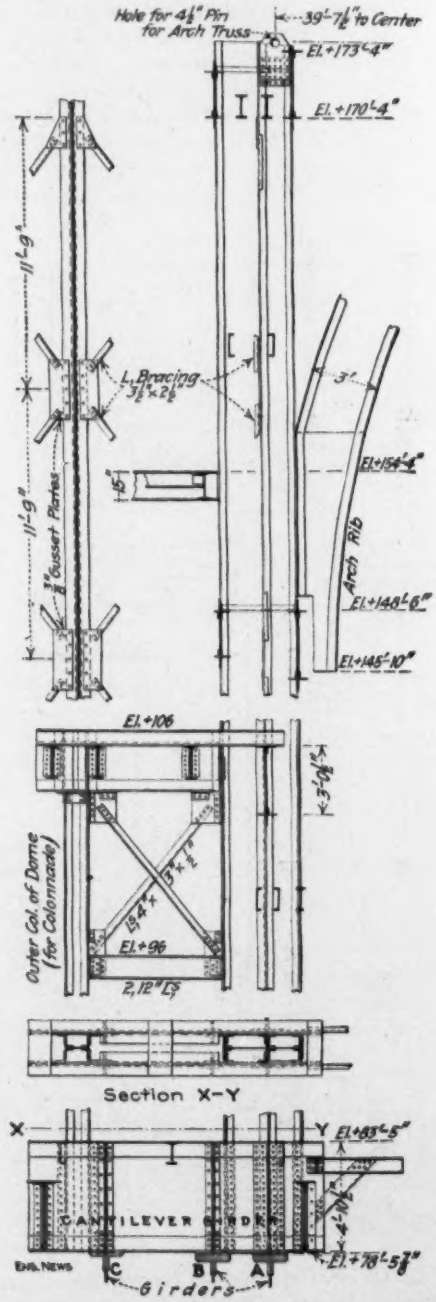

Fig. 7. Cantilever Girders and Columns of Barrel of Dome

webs of the distributing girders have stiffening angles on the inner faces, and transverse diaphragm plates are riveted between some of these stiffeners, as shown. Each anchor column (Fig. 4) is of H-section, composed of four angles 6x4 in. and a web plate 12x½ in. Its shoe has four 1-in. anchor bolts 20 in. long, fitted to an anchor plate 27x34 in.

The 8-ft. girders of the ring course have ½-in. webs and four flange angles 8x8 in. In the shorter girders (in pairs) the webs are the full depth of 96 in. In the triple sets of the longer girders, the two inner ones have 90-in. webs and four ¾-in. cover plates on each flange

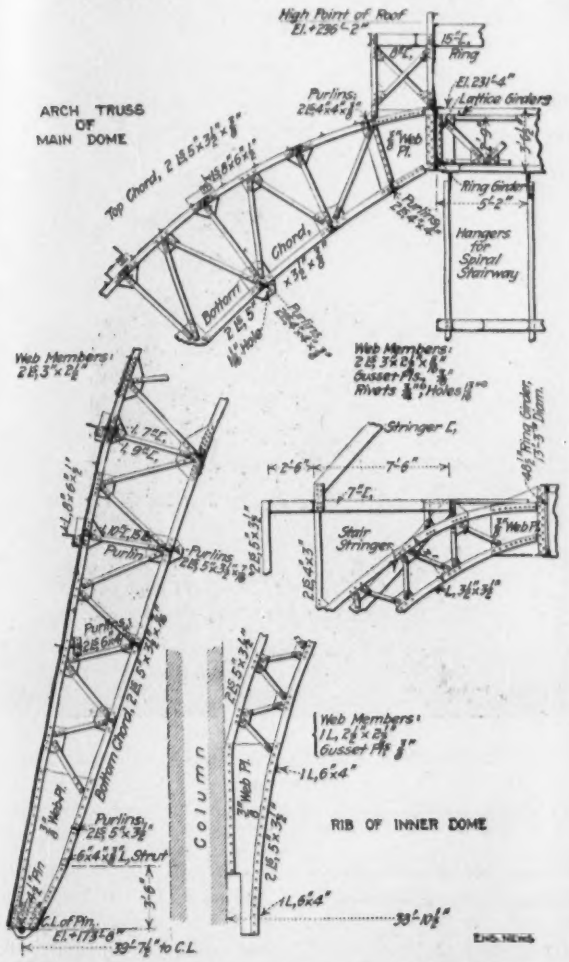

FIG. 8. DETAILS OF ARCH-TRUSSES OF MAIN DOME, AND TRUSS RIBS OF INNER DOME

(from full length to 21 ft.), while the outer girder (carried by the anchor columns) has a 94-in. web and one cover plate on each flange. The struts between the main and anchor columns are 24-in. plate girders.

THE DOME

The construction of the dome, including its barrel or base portion which rests upon the steel supporting tower, is shown in Fig. 5. Across the ring course of double and triple plate girders, which forms the top of the substructure or tower, are laid 24 cantilever girders in radial position, and having a projection or cantilever length of 5 ft. There are four of these on each long side and two on each short side of the octagon, arranged as shown in Fig. 6. The inner ends of these cantilever girders carry 24 columns about 90 ft. high (to Elev. 175), which support the trusses of the dome proper. The outer ends of the girders carry 24 columns extending to Elev. 106, and supporting the grillage under the stone colonnade which surrounds the barrel or base portion of the dome (Fig. 5). The inner or main columns are heavily braced, and to their inner and outer faces are attached pairs of 3-ft. plate girders which form rings supporting the masonry walls at elevations 106, 135, 149 and 174 ft. The tops of these inner columns have pin plates for the 4½-in. shoe pins of the dome trusses.

These dome trusses are in radial position, with their lower ends carried by the shoe pins in the heads of the columns and their upper ends riveted to the web of a plate-girder ring 10 ft. 4 in. in diameter, at Elev. 231. Above this rises the steel framing of the lantern. The outer shell of the dome has a horizontal radius of 80 ft. at the spring line. The vertical outer curve (on the chord of the arch trusses) is struck from two radii; one of 50 ft. 4 in. at the lower portion and the other 32 ft. for the upper portion.

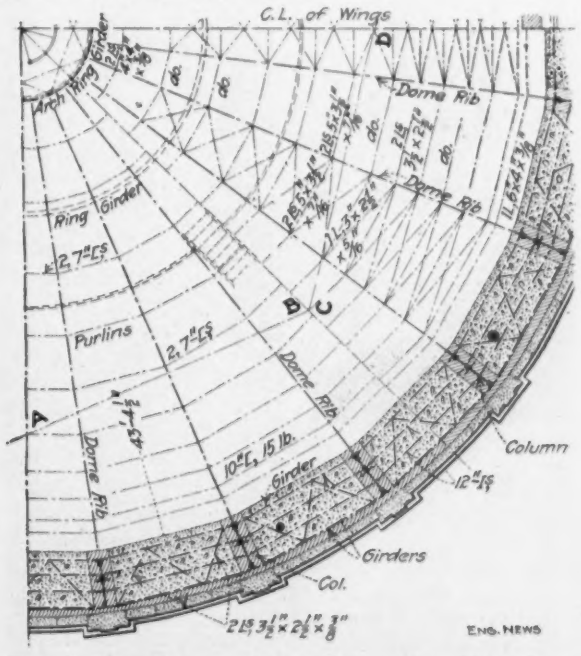

FIG. 9. PART PLAN OF STEEL FRAMING OF MAIN DOME

A-B. Plan showing purlins on top chords.
C-D. Plan showing purlins and bracing on bottom chords.

Within this main dome is an auxiliary or ceiling dome, which is the one visible from the floor of the rotunda. This is carried by 24 radial lattice-web ribs, having the lower end riveted (by the back of the outer chord) to the faces of the columns, at Elev. 146. The upper ends are riveted to the web of a ring plate girder 26 ft. 6 in. in diameter, at Elev. 180. The vertical radius on the inner chord of these ribs is 40 ft. 5 in. for the lower portion, and 15 ft. 9½ in. at the top. About 20 ft. above this circular opening or "oculus" is a concave ceiling about 34 ft. horizontal diameter, the framing of which is suspended from the ring girder at the summit of the

main dome. Above this inner dome are stairways leading to a spiral stairway from the ceiling to the lantern.

The exterior of the main dome is sheathed with terra-cotta, and has an exterior covering of granite in blocks about 8 in. thick. This exterior surface is relieved by heavy radial ribs and recessed panels. The interior of the inner dome carries the coffered and decorated ceiling or lining.

In design, the outer dome shell was not treated in accordance with the true dome theory, but each pair of opposite truss-ribs was considered as a three-hinged arch. The tangential peripheral stress resulting from the horizontal component of the thrust from the ribs at the top of the dome columns was taken up by tension in the 2-ft. and 3-ft. plate-girder rings framed between these columns. The thrust at the summit or eye of the dome was taken into the 42-in. ring girder, which is braced on the inside by a double system of lattice girders forming two squares, the corners of which intersect the ring girder at the connection of every third rib. The angle flanges of this lattice-girder bracing are so turned as to allow one system to pass through the other and permit of riveted flanged connections of one to the other. The loads assumed were 175 lb. per sq.ft. for the exterior stone sheathing, and 30 lb. per sq.ft. for wind and snow.

FIG. 10. STEEL ERECTING TOWER WITH TWO ELECTRIC DERRICK HOISTS

The circular sky dome or ceiling of the inner dome is suspended by hangers from the ring girder of the main dome, and this same arrangement is used to carry the spiral stairs and landings. The inner dome itself is self-supporting, with each rib connected at the lower end to the main dome columns and at the upper end to a ring girder at the crown. The rib thrusts considered in the design of this ring girder were those which were found to be a maximum due to the combination of live load on the annular platform (at Elev. 184 ft. 3 in.) and the dead load on the ribs as applied to two opposite ribs considered as a three-hinged arch. These ribs were designed as fixed at the columns and the ring or crown girder, and carrying the maximum live and dead loads due to a symmetrical loading.

STRUCTURAL FEATURES OF THE DOME

The cantilever girders which carry the columns of the dome, and which are placed in radial positions across the girders of the ring course at the top of the tower (Figs. 3 and 6) are shown in Fig. 7. They are about 13 and 22 ft. in length. Each has two $\frac{5}{8}$-in. webs 58 in. deep and eight flange angles 8x6 in., while the longer ones have also two cover plates on each flange. The inner or main columns of the dome (Fig. 7), resting upon the inner ends of the cantilever girders, have each a 41-in. web plate and six angles 6x4 in.; from the base to Elev. 135 there are also two additional angles (making four at the middle of the web plate) and also a cover plate ac[ross] each flange. To the top of the web plate are riveted [pl]ates or jaw plates for the shoe pins of the truss r[ib]s. The outer columns (to support the colonnade or circul[ar] row of stone columns) consist each of a 12-in. web pl[ate] and four flange angles 66x4 in.

The details of the arch-trusses of the main dome a[nd] the ribs of the inner dome are shown in Fig. 8, wh[ich] Fig. 9 shows the bracing between the arch-trusses, bo[th] on the outer and inner chords. The arch-trusses of th[e] dome have chords of two angles 5x3½ in. with web mem[-] bers of pairs of angles, and a web plate at each end. O[n] the outer side, there are purlins, each composed of tw[o] 7-in. channels (with the flange of one fitted against the

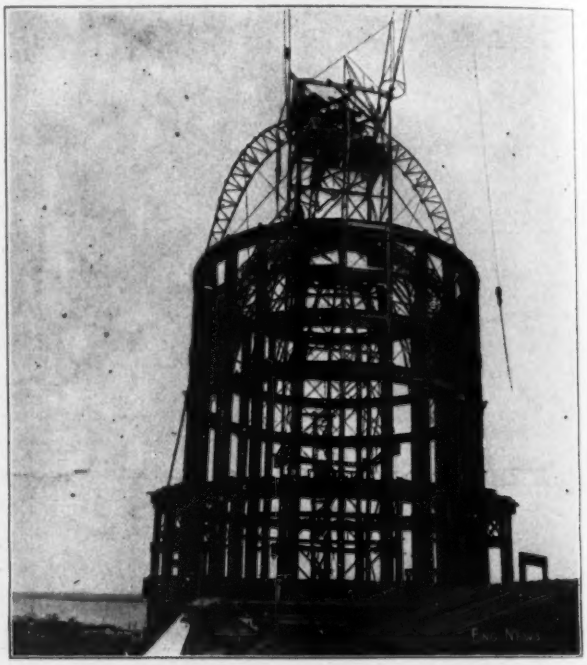

FIG. 11. ERECTING THE ARCH TRUSSES OF THE MAIN DOME

(Through the steel tower may be seen the truss ribs of the inner dome.)

web of the other); also a series of rings of 3x3-in. tee-bars riveted to the chords of the trusses. On the inner chords, the purlins are pairs of 4x4-in. angles. The ribs are braced in pairs on the inner chords, and there are two lines of continuous sway bracing. The heads of the trusses are riveted against a 42½-in. ring girder, 10 ft. 4 in. in diameter. On the inner side of the inner chord and the outer side of the outer chord is a sheeting of 4-in. terra-cotta blocks, with a waterproofing course on the back of the inner sheeting. The granite rib blocks of the exterior covering are held by angle irons riveted to the dome framing (Fig. 5).

The ribs for the inner dome have the chords composed of pairs of angles 4x3½ in., with web members of single angles 2½x2½ in., and a web plate at each end of the rib. The purlins are on the inner chords only and consist of angles 6x4 in. and 3½x3½ in. The tops of the ribs are framed against a ring girder 48½ in. deep, having a diameter of 26 ft. 6 in. In both sets of ribs

the connection plates or gussets are ⅜ in. thick and have ¾-in. rivets in $1\frac{3}{16}$-in. holes.

Material and Erection

All structural material is of openhearth steel, conforming to the specifications of the American Society for Testing Materials. The painting consists of one coat of "Nobrac" at the shops, and two coats of "Nobrac" after erection. The Modern Steel Structural Co., of Waukesha, Wis., built and erected the structural work. T. McCarthy, of Madison, Wis., has the general contract for the masonry work. This comprises the foundations, the

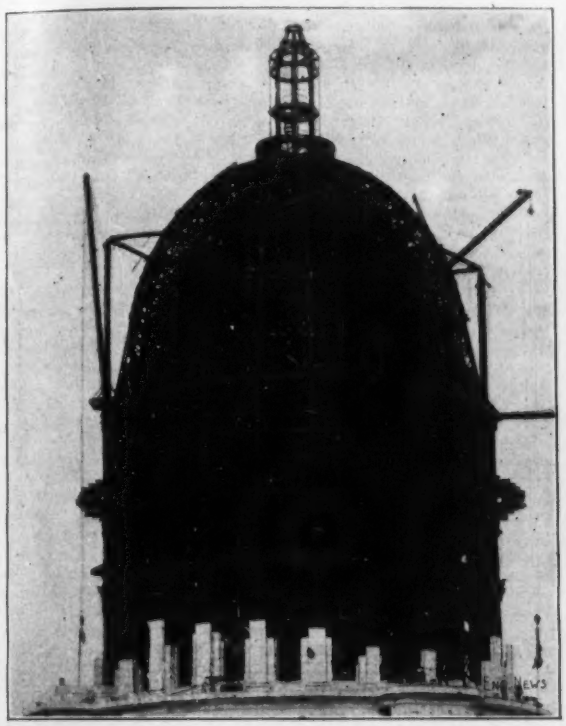

Fig. 12. The Completed Steel Framework of the Dome of the Wisconsin State Capitol, Madison, Wis.

large concrete piers and the backing of the granite work which is furnished and set by the Woodbury Granite Co., of Hardwick, Vt.

The erection of the steel work of the dome was handled by two electric derricks, the booms of which were attached to the corners of a large steel erecting tower at the vertical center line of the dome. This tower had a base about 30 ft. square, and extended from the top of the foundation piers to the level of the ring girder at the center of the outer or main dome trusses. This tower with its booms is shown in Figs. 10 and 11, the latter showing the erection of the arch-trusses of the main dome. The erection of the granite work of the dome was handled by four electric derricks. In each of these, the mast rested on one of the lines of horizontal girders, and its head was supported by tie timbers anchored to the framing of the dome (Fig. 12).

There was no general contractor for the entire work. Separate contracts for the several classes of work were awarded by the Capitol Commission, and the entire construction was under the supervision of the architects. This construction work is under the direction of John H. Gormley, Resident Superintendent for the architects.

Chicago Harbor and Subway Plans

The Chicago Harbor and Subway Commission is an engineering commission, intrusted by the City Council with the general supervision of plans for a municipally-owned outer harbor, and a system of municipally-owned passenger subways. The Commission was organized in December, 1911, and up to the present its activities have covered preliminary plans only, as in both the harbor and subway projects the necessary legislation for actual construction has been delayed pending the settlement of matters of public policy. It is composed of John Ericson, E. C. Shankland and J. J. Reynolds. Wm. J. Shanks is secretary.

The Passenger Subways—The Commission states that there is no prospect of construction work until after next April, when a referendum vote will be taken upon alternative methods. One plan is for a limited downtown subway system, to cost about $17,000,000; the other is for a comprehensive, city-wide subway system, to cost about $70,000,000.

The Outer Harbor and Piers—The voters of Chicago in 1912 authorized a $5,000,000 bond issue for the proposed outer harbor, but further legislation was needed to quiet disputed titles along the lake-front strip that the city proposed to use. The recent legislature gave the city supplementary powers to clean up these titles, and this new harbor act went into effect July 1, 1913.

The city now proposes to pay the Chicago Dock & Canal Co. $300,000 for its claimed littoral rights on the strip to be used in building the first pier, which will be in line with Indiana St. The Harbor and Subway Commission is now preparing specifications for contract bids for this pier, and bonds to cover construction expenses will be sold by the city as the work progresses. This first pier will be for passenger and package-freight service. It will be 3000 ft. long, and 292 ft. wide, with its south line 700 ft. north of the Chicago River. A second pier, for freight traffic exclusively, will be built later between the first pier and the river.

There will be an 80-ft. roadway in the center of the new pier, with sheds 100 ft. wide on each side and outer 6-ft. spaces. The building construction will be of fireproof material. The superstructure will have a second floor to be used for passengers during the summer season. This will have doorways 100 ft. apart, and from each doorway a passageway will slope to the edge of the gallery, from which gang planks can be extended to vessels. Before and after the summer passenger season, the second floor of the pier will be used for freight storage.

The outer 645 ft. of the pier will be for vessels with an exclusively passenger traffic, and here the upper deck (and part of the lower deck) will be reserved for recreation purposes, with a large rotunda for band concerts and other purposes. Street railway cars will be looped at the shore end of the pier and provision is made for a line of street cars along its second floor.

The U. S. Government has made an appropriation for extending the present north outer breakwater to protect the piers, and $300,000 of this will be available when harbor construction has been begun by the city.

Efficiency of Coagulating Basins at St. Louis, Mo.*

By W. F. Monfort†

Shortly before the World's Fair at St. Louis, as an emergency measure, coagulation with iron sulphate and partial softening with lime were added to the system of plain sedimentation which had been in use for years. The scheme made such a manifest improvement in the appearance of the effluent that it was accepted by the local public as the last word in water purification. At times the effluent has been excellent in every way; again of doubtful character. It is only within the past year that the suggestion has been entertained to supplement with a filter plant the abridged form of treatment now in use. [A mechanical filter plant is now under construction.—Ed.]

The clarification plant of the St. Louis Water-Works comprises a coagulant house ample for treating with lime and iron sulphate more than 160,000,000 gal. per day, with a series of unbaffled basins. There are six basins of about 25,000,000 gal. capacity, one of 40,000,000 gal. and two of 30,000,000 gal. each, through all of which in series the treated water flows. The combined capacity of the system is 230,000,000 gal.; the combined area, about 52 acres.‡

It has been popularly assumed that the plant as now constituted is ample for adequate treatment of an indefinitely large volume of water. One writer estimates the capacity of the plant at 250,000,000 gal. per day. Operating results warrant a much lower estimate based on the degree of success attained in treatment of varying quantities of different waters, ranging in suspended solids from 14 to more than 8000 parts per million.

There is reason to believe that even in the first year of operation the clarifying system was overtaxed. The records of this laboratory afford but twelve determinations of suspended solids in river and clear well samples during the year 1904-1905, of which two must be discarded.

Treatment was begun on March 22, 1904. The first recorded result nine days later suggests the inadequacy of the plant, which then comprised but the six basins in series, to clarify 67,000,000 gal. per day when the suspended matter was 2400 p.p.m. For 97.5 p.p.m. of suspended matter in the finished water indicates the introduction into the distribution system on the corresponding day, April 1, 1904, of 27.5 tons of suspended solids. It is possible that this result was abnormal, since mud previously deposited in conduits and clear well may have been loosened by the flow. The second result, two months later, shows improved working, but suggests that 75,000,000 gal. could not be handled properly when the suspended solids in the river exceeded 1700 parts. According to present standards the only passable results in the first twelve months of operation are those of Nov. 30, 1904, and April 1, 1905, when river solids were less than 1000 parts and the daily consumption was under 80,000,000 gal. Under other conditions the finished water contained notable amounts of solid matter. High consumption during the World's Fair period overtaxed the new clarification plant the entire summer. Still these results show a vast improvement over conditions in the year prior to introduction of the lime-iron method, when the public was expected to make no complaint when 450 p.p.m. of suspended matter appeared in the tap water.

In the second year of operation results were generally better. However, of 130 recorded determinations of suspended matter in clear well samples, 40 gave negative results. The figures given in Table I are averages of 130 determinations for the river and 90 for the clear well. The average daily consumption for each month is from high service pumping.

TABLE I. WATER CLARIFICATION RESULTS AT ST. LOUIS, 1905–6

1905–1906	Average suspended solids parts per million: River	Clear well	Average daily consumption million gal.
April	....	....	67.0
May	1962	12.7	69.5
June	1809	2.7	78.4
July	2845	7.4	75.8
August	1950	12.0	77.9
September	2056	11.0	73.7
October	1015	5.8	68.7
November	603	5.2	64.0
December	386	9.0	63.0
January	611	9.2	62.6
February	649	7.1	65.2
March	641	7.7	62.2
	1320	8.2	69.0

Compared with the new standard set by the previous year, the results for 1905-6 were good. Apparently lower consumption favored better operation; although the occurrence of so many impossible results (negative quantities) makes interpretation difficult. The records show that from June to January, inclusive, high caustic alkalinity was carried in the treated water. This is perhaps the explanation of the high suspended solids recorded, which seem to indicate somewhat less than 80,000,000 gal. per day as the maximum capacity of the plant when the river was carrying 1800 p.p.m. of suspended solids.

The writer specifically disclaims any responsibility for the foregoing results of operation and analytical data. Results for subsequent years were determined under his direction and are believed to represent with a fair degree of accuracy the working of the plant. Averages are used in lieu of rehearsal of the full detail of determinations made on all save holidays. Inasmuch as total displacement of water in the entire basin system requires from seven to fifteen days, the averages by months of operation better represent the blended waters issuing from the clear wells to the distribution system. They are platted in the accompanying diagram, in sequence with those of 1905-1906.

The diagram shows graphically that when suspended solids in the raw water were below 1500 p.p.m. and consumption did not exceed 75,000,000 gal. per day, the average suspended solids in the clear well water were usually not more than 1 or 2 p.p.m., increasing slightly with a rise in either river solids or consumption, and greatly with concurrent rise in both.

It is noticeable that when pumpage was much above 75,000,000 gal. and river solids greatly exceeded 1500 parts the quality of the treated water was seriously changed for the worse. In general the curves of suspended matter in the treated water reflect the effect of high-river solids and high consumption, following one or both in extreme cases, and illustrating the fact that these uncontrollable factors produce conditions which the present plant cannot meet. It appears that a raw water carrying not more than 1500 parts of suspended matter can be made acceptable to the public of St. Louis so long as consumption does not exceed 75 to 80 million gallons per day, but that the capacity of the plant varies inversely with the suspended matter.

The diagram further shows that from 1905 to 1910 consumption exceeded 75,000,000 gal. per day only in the summer and fall months, while in 1911 and 1912 the average daily consumption has rarely fallen below this figure at any time. The rise in river stage caused by rains of the late spring and summer brings high average suspended solids in the raw water; for suspended solids vary with the stage. Coincident with these times of high turbidity comes the heaviest draught upon the distribution system, when lawns are to be sprinkled and the greatest waste of water occurs. It is unfortunately true that periods of highest turbidity are generally periods of greatest consumption.

EFFICIENCY OF CLARIFICATION

There is abundant evidence that efficiency of clarification and bacterial reduction are conditioned by

Rate of flow through basins.
Amount of sludge already deposited.
Character and quantity of suspended solids in the raw water.
Temperature of water in river and basins.
Wind velocity and direction.

When a water properly treated is passed through the settling basins, 97 to 99% of the suspended matter is precipitated in the first basin, the percentage removal depending upon the character and quantity of solids contained, the temperature, wind velocity and direction, and the amount of sludge in the filling basin. In passing succeeding basins the remaining suspended matter undergoes a further reduction of one-half or one-sixth, corresponding to a few hundredths of 1 per cent. reckoned on suspended matter in the raw water, likewise dependent upon velocity, size of particles, wind and temperature. The major portion of clarification is, however, accomplished in the filling basin of 25,000,000 gal. working capacity. There is no provision for applying chemicals after the water enters the basins. The efficiency of the plant, therefore, depends primarily upon the volume of water which can be satisfactorily clarified in the filling basin.

The weight of suspended matter in the effluent of the successive basins varies with the weight of solids carried by the raw water. The percentage is approximately constant; the actual weight of solid matter remaining in the finished water is proportional to that originally present in the river water.

Since the course of the currents in the basins will vary with changing rates, and the velocity and consequent carry-

*Abstract of a paper read before the Illinois Water Supply Association, early in 1913. Some of the tables are omitted.

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‡See "Eng. News," Jan. 11, 1912, for description of this plant.

ing power of currents increases with increase in pumping, the weight of suspended matter carried over the first weir is subject to wide fluctuations. A change in pumping from a rate of 60 to 90 million gallons per day has increased the suspended matter in the treated water (clear well) by from 2 to 7 p.p.m., and a further sudden increase to 120 millions per day has caused a further rise of 10 to 13 parts, which was carried through the entire series of basins and conduits to the clear well, where 25 to 30 parts of solids in suspension occurred.

Slight changes in temperature suffice to alter the course of currents through the basins. In the spring, when the temperature of water in the river and basins is rising, the sludge is less subject to disturbance than in the fall and early winter, when, with falling temperature, the influent water, more dense than the warmer water of the basins, passes downward over the sludge, causing it to carry over weirs.

In the fall, with lowering atmospheric temperatures, the sludge and water in the bottom of the basins are sometimes 1° F. or more warmer than the surface water of the basins and river. Circulation is then effective in changing the course of influent water currents, making them deeper and increasing the scour.

The sludge is further subject to disturbance by wave action when high winds prevail, a frequent occurrence in March. In such a case the amount of suspended matter carrying over the weir from the filling basin shows a marked increase. Our basins have a working depth of about 14 ft. Similar effects of wave action have been noted in other reservoirs 18 ft. deep.

The basins at the Chain of Rocks (52 acres) are all uncovered, all used in series, and therefore, subject to disturbance by each of the agencies affecting their successful working.

CHARACTER OF SLUDGE

Suspended matter, with the coagulum produced by chemical treatment, subsides rapidly, undergoing a change in volume during its accumulation in the bottom of the basins. When freshly formed it is loose, disseminated through the full volume of water in which it forms; under average conditions after an hour and a half it occupies about 3% and after 24 hours about 2% of the original volume. After this lapse of time only the newly precipitated portions are disturbed by gentle currents.

Opening mud gates at 8-hour intervals seems to reduce the sludge only near the gates, since it follows in a general way the contour of the bottom of the basin and is of such consistency that it does not flow readily over the compact material of earlier subsidence. The tendency is for each new deposit to collect more thickly upon the highest points of previous deposits.

BACTERIAL REMOVALS

Bacterial purification is proportional to the degree of clarification, falling a little below the percentage removal of suspended solids (see Table II) for the reasons which are cited below.

TABLE II. REMOVAL OF BACTERIA WITH SUSPENDED SOLIDS IN THE ST. LOUIS COAGULATING AND SETTLING BASINS

	Suspended solids		Bacteria	
	parts per million	per cent. removal	per c.c.	per cent. removal
River	1444	....	57,000	.....
Basin 1	14.0	99.01	933	96.1
Basin 2	12.1	99.16		
Basin 3	8.4	99.4	500	99.1
Basin 4	7.1	99.5		
Basin 5	5.8	99.6	100	99.8
Basin 6	5.6	99.6		
Clear well	2.6	99.8	42	99.99

Bacteria entangled in the natural sediment of the water, and gathered into coherent masses with the coagulum, concentrate in the sludge to the extent of a million or more per cubic centimeter; are subject to disturbance and dissemination through basin contents by varying currents, however produced, and are easily carried through later basins to the clear well.

It has been observed that slight changes in temperature of the influent water, causing almost infinitesimal differences in the density from that of water at different levels in the basins, give rise to turbid, polluted effluents from each basin in series, as the less compact layers of sludge are moved; that overturning, which in large bodies of deeper water occurs but twice a year, may occur several times a week when warm and cold days alternate in the spring and fall; and that with a slight rise in suspended matter carried from the older sludge in the filling basin may come an altogether disproportionate increase in bacteria, by reason of alterations in the rate of pumping, or the currents produced by high winds sweeping along the surface of a half mile of water. When it is considered that the combined surface of water exposed to winds and temperature changes is more than 52 acres it will not seem idle to refer to what might at first appear as entirely negligible factors.

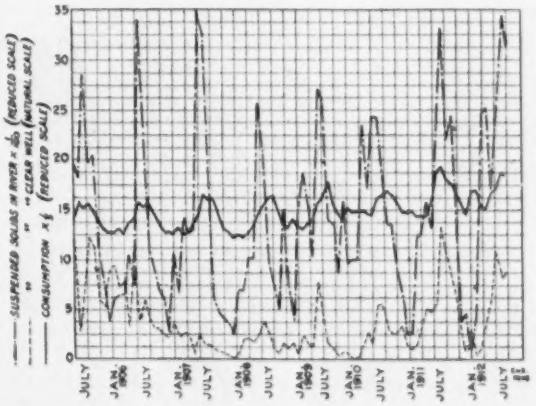
An abrupt change in the rate of flow through the basins may cause the sludge to carry its burden of bacteria through successive basins.

A change in the direction of flow incident to cleaning and restoring a basin to service affects both suspended solids and bacteria per cubic centimeter in the finished water. Basin 1 was thus put in service June 6, 1912. The rise of bacteria in the clear well samples was from 150 per c.c. on the 6th, to 3300 in the following week: [99.6 to 92.9% removal as compared with river water.—Ed.] In this case irregular pumping was a factor in producing bad results.

Bacterial reductions are subject to disturbance from too many factors to give constant results. We have no safeguard against turbid, contaminated water under these conditions.

River samples usually give evidence of the presence of bacteria of the B. coli group in $^1/_{100}$ c.c. The finished water has given counts of more than 30,000 bacteria per c.c. on gelatin at 20° C., and members of the B. coli group have been found in 6% of tests on 1 c.c. samples in a single month.

In reviewing bacteriological results for past years there is



RELATION OF RATE OF WATER CONSUMPTION TO CLARIFICATION AT ST. LOUIS, MO., 1906 TO 1912

noticed a very wide divergence at any one sampling point, and extremely irregular counts for a given period at various points in the clarification system. The utmost we can hope for is a large percentage reduction of bacteria. We can have no assurance that the water which enters the distribution system will be free from pathogenic organisms regularly. While the improvement in the character of the effluent since the introduction of the clarification scheme seems to have reduced the typhoid death rate, the quality is still far from that of a good filter plant.

RESIDUAL SOLIDS IN DISTRIBUTION SYSTEM

Water leaving the clear well contains small quantities of suspended and dissolved iron compounds, small particles of calcium and magnesium compounds, and larger quantities of silt and siliceous matter too fine to be deposited during rapid flow through the sedimentation system. The amount of this material daily introduced into the distribution lines during the first year of operation, calculated from suspended matter and daily consumption, was as high as 27.5 tons, averaging 8.7 tons per day for ten analyses.

TABLE III—AVERAGE WEIGHT OF SUSPENDED MATTER IN ST. LOUIS WATER-SUPPLY BY YEARS

	Solids in tons	Consumption, million gal.
1904–1905	8.7	79.0
1905–1906	2.35	69.0
1906–1907	1.05	70.1
1907–1908	0.31	68.9
1908–1909	0.5	70.7
1909–1910	0.34	75.4
1910–1911	0.92	76.1
1911–1912	1.74	83.5

This material is intermittently discharged from taps over the city in a very irregular way. It is a matter of common observation that after unusual draft upon the mains in a portion of the system, very high turbidity appears, local, or affecting large sections of the city, according to the degree of the disturbance. Following a large fire complaints of turbid water are very numerous.

INCRUSTATION IN DISTRIBUTION SYSTEM

Because ours is a partially softened water calcium carbonate is always present in the finished product. The softening process is completed slowly at summer temperatures and in winter is incomplete even when the water passes to the distribution system. There is, therefore, more or less deposit of calcium carbonate in the mains and service pipes. Even with high bicarbonate alkalinity in the filling basin the water leaving the sixth or ninth basin is still supersaturated with calcium carbonate.

Connection with a 7-ft. steel flow line was made in January, 1908, and the city supply drawn through it for 74 days. Examination at the end of the period disclosed a deposit, principally of calcium carbonate, $\frac{1}{32}$ in. thick when moist, which shrank to $\frac{1}{64}$ in. in drying. The water had an average total alkalinity of 59 p.p.m., of which 25 were due to neutral carbonates and 34 to bicarbonates. Temperatures ranged from 32 to 47° F.

During the earlier years of operation there were notable deposits in meter gears, fish traps and the like.

While the deposit in the distribution system does not now seem to be increasing rapidly, there is still some incrustation in progress, due to the blending of unequally softened waters. Trouble from this source can be lessened by longer storage, which will equalize the quality of the water before passing the high service pumps—a very costly expedient; or by so regulating the degree of softening that the finished water shall show a high degree of uniformity—a difficult matter when the raw water is changing quickly. Lower regular velocity through the settling basins would allow longer time for softening reactions; this entails the use of several filling basins, instead of but one with the rest in series. Finally, further reduction of this trouble could be effected by changing the order of chemical treatment, adding lime first and agitating the treated water before the charge of iron sulphate is applied.

GENERAL SUMMARY AND CONCLUSIONS

During the first year of operation at St. Louis the weight of solid matter carried into the distribution system averaged 25 p.p.m.—ranging from none to 97 parts; the reduction was about 98% reckoned on the weight of solids in the raw water. While this was a decided improvement over conditions prior to treatment the introduction of an average of 8.7 tons of suspended matter per day into the mains left much to be desired if one compares this result with the standard set by a satisfactory filter plant, whose effluent contains no weighable quantity of solid matter in suspension, and shows no visible turbidity or opalescence in bright sunlight.

The local standard of purity has advanced from year to year since the clarification system has been in operation. Prior to 1904 sedimented water containing 60 to 450 p.p.m. of solids in suspension was accepted, if not approved. Compared with that standard the quality of water furnished in 1904-1905 was excellent. The next year showed a marked improvement, and established in turn a standard for comparison of the succeeding year's supply. With each subsequent year the quality of water furnished has been progressively better, until 1910-1911, when consumption so far exceeded the plant's capacity that there was a falling off in the quality of the effluent, although it was still superior to that supplied before 1907-1908. The capacity of the present plant must be increased. The extensions and changes made must provide a better effluent than the present public has been educated to demand.

It is apparent that the addition of a filter plant to the coagulating basins is essential if the residual sediment is to be finally removed, and the high color which sometimes characterizes the raw water is to be reduced to an acceptable degree. Operations under prevailing conditions in a plant of this size and character do not admit of the close control possible in a filter plant, where the units (filters) are small, subject to immediate supervision and washing, and their output regulated by rate controllers. Furthermore, with filters the solid matter collected with entangled bacteria is quickly and permanently removed from water passing through them; whereas coagulating basins give too frequently only a temporary separation. It is manifestly impossible to interrupt the flow through any one basin at will, should the water in it prove unfit for use, without seriously affecting the contents of other adjacent units by altering the course of currents through them. Nor is it possible to wholly eliminate the previously accumulated sludge in the filling and sedimentation basins, so that after cleaning the first water passing through them shall be faultless.

The abridged form of treatment adopted prior to the World's Fair has beyond question served a very useful purpose. The question has arisen whether the present system of partial softening, coagulation and sedimentation shall be extended by adding further sedimentation basins to the already existing plant, or supplemented by a filter plant which shall afford a perfectly clear water at all times, with fairly constant bacterial removals and the possibility of immediate control of operating conditions. The cost of constructing two basins of 40,000,000 gal. capacity each was about $3.50 per 1,000,000 gal. capacity.

It is hardly possible to make the necessary changes in the present basins and add the required reservoir capacity to provide for clarification of the volume of water which will certainly be consumed within the next ten years at a cost of less than $1,500,000. Whereas a filter plant added to the present basins, with certain changes in the basins, comprising 40 filters of a normal rating of 4,000,000 gallons each in 24 hours, can be constructed at a cost of about $1,250,000.

There are locations where basin construction is cheaper, where prolonged sedimentation in very large reservoirs may be advantageously used in producing an acceptable effluent for a time—until the public forgets its earlier satisfaction with an improved water supply, and clamors for further betterment. Before such installations are begun there is need of very careful study of the quality of the untreated water, to show its adaptation to coagulation methods and good judgment as to arrangement of the plant with a view to the ultimate addition of filters.

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Some Necessary Precautions in the Design and Supervision of Construction of Low-Truss Steel Highway Bridges

By G. M. Braune*

The plans for small country highway bridges are usually made by bridge company agents, and experience proves that the majority of county commissioners will accept most any plan so long as the price is low enough. A great number of these bridges are indeed doing service today for ordinary light traffic, but should the members be stressed up to the amount for which they were "designed" the chances are that they will refuse to carry the loads. The peculiar thing about the situation is, moreover, that the collapse of such structures, and even the publication of these occurrences, seems to have small effect upon the future construction of similar structures.

The writer has knowledge of the collapse of a bridge very similar to that at Wellsville, Colo., which was described in Engineering News, Mar. 13, 1913, p. 532. This bridge was built in one of the Southern States some years ago. The top chord was made of two angles and a web plate, in the form of a T-section. In this case, however, the bridge had been accepted by the county commissioners, and it was not until a few days after the acceptance that it collapsed under a drove of horses.

Precautions When Using a Heavy Concrete Floor—Concrete floors are rapidly replacing the former light wooden floors; therefore the actual dead-load stresses occurring in the bridge members are a large percentage of the maximum computed stresses, and the details are seldom designed to properly take care of these large actually occurring stresses. The design of the details is too often turned over to a young, inexperienced draftsman, whose knowledge about the distribution of stresses is generally somewhat meager.

Not very long ago the writer was shown the shop drawings for a 75-ft. pony truss "designed" to carry a heavy concrete floor. A splice of the bottom chord (made of $\frac{5}{8}$-in. material) was attempted at one of the lower panel

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points, and the draftsman had intentionally, although ignorantly, neglected to provide sufficient splice material to equal the area of the cut chord. The correct number of rivets had, however, been provided to take care of the stresses in the different members that were attached to the ⅜-in. gusset plates. When the error was called to the draftsman's attention, he was much surprised, and informed the writer, "that was their usual method of designing splices." Luckily the bridge had not been shipped, and the corrections were made in the shop.

Since heavy concrete floors have come into fashion, resulting in large dead-loads and large reactions, it is very essential that the ends of the trusses should be provided with pin bearings, although this detail is to a great extent ignored, not only by a number of bridge companies, but also in some of the recent textbooks. Owing to the uneven distribution of the reaction over the bearing plate, large secondary stresses will occur in the members designed to take only direct stress.

The writer is personally acquainted with an accident which happened some years ago to a heavy low-truss bridge in which this detail was omitted. As the bridge was not provided with end-pin bearings, large bending moments occurred in the bottom chord, and also, of course, in other members of the truss. One day, when a heavy electric car was crossing the bridge, the bottom chord parted near the end-panel point. Computations showed that the member was carrying a conservative direct working stress; and faulty material was not the cause of the failure as was shown by subsequent tests.

Low-truss bridges, 100 ft. long and over, have been constructed in Europe and in this country, which, from a standpoint of strength, stiffness and æsthetics are all that could be desired. The computations of the stresses, makeup of the sections, and the design of the equally important details, require, and always will require, however, a certain amount of intelligence and experience.

Points about Some of the Details—The lateral forces acting upon the top chord of a low-truss bridge, whether from external causes, or resulting from column action, can be cared for without the addition of unsightly floor-beam knee-braces by designing the vertical posts or diagonals of a form (say, two angles and a web plate, or if the design requires it, four angles and a plate) capable of resisting bending and shear. The overturning effect of the lateral forces can then be transferred into the floor-beams by riveting the vertical posts to the floor-beam web, or through the diagonals into the gusset plates, and thence by means of a diaphragm, into the floor-beams.

The bending moment in the verticals and diagonals, and hence the resulting indirect shearing action on the rivet connections, would be obtained by multiplying the assumed lateral force by the distance between the center line of the top chord (position of application of lateral forces) and the center of the group of rivets connecting the floor-beam to the vertical posts. The rivet connections should be sufficiently strong to take care of direct shear and also the shearing action resulting from the bending in the posts and diagonals.

The top chord should be of an inverted trough section, sufficiently wide to allow the vertical posts and diagonals to be inserted between the gusset plates. Pin-connected pony trusses should not be tolerated.

Supervision of Construction—As long as intelligent supervision is not had over the design and construction of highway bridges, we will hear of occasional collapses; and there will also occur numerous failures which will not be brought to the notice of the engineering profession.

The question before us is, how can an improvement of this state of affairs be brought about? Some engineers honestly believe that by licensing the practice of engineering good results could be secured in this direction; others hold, however, that this would not overcome the evil, on the other hand, it might make matters worse; for an engineer holding a license would be *per se* in the eyes of a great many county commissioners a competent bridge engineer, whereas we all know that this would not be the case.

Proposed Legislation

A better method would be the enactment of some uniform legislation by all the different states, covering the design, specifications and supervision of all bridges to be built, excepting municipal bridges, which would be built under the supervision of the city administrations.

No doubt it would be somewhat difficult to convince the majority of our state legislators that the service of a competent and experienced bridge engineer is an absolute necessity for the protection of the taxpayers' interests in the construction of their bridges; but it might be accomplished by the concerted action of a number of well known and competent engineers, or by some action of the American Society of Civil Engineers.

It would require the presentation of a report containing convincing data that would effectually show the law makers the need of such legislation. Perhaps the collection of accurate data covering the failure of improperly designed highway bridges throughout the country and their presentation in the form of a report to the state legislatures by a committee composed of prominent engineers might have the desired effect.

After the passage of such legislation the selection of the proper official to render efficient service would be as important as the laws themselves, and to secure this end a competent bridge engineer (not a political engineer) should be appointed by the governor of each state. The selection could be made from a list of names submitted by the American Society of Civil Engineers or some other recognized professional organization of engineers. The salary paid for his services would be a bagatelle as compared to the good resulting to the taxpayers; and in a few years the results of honest supervision would be highly appreciated by them.

An objection to this plan might be raised by some engineers, claiming that the enactment of such laws and the appointment of a state bridge engineer would curtail their practice. However, a large percentage of the bridges are small, costing not more than $1500 to $2000; and from experience it is found that the county commissioners are not willing to pay the price demanded by an honest and competent private engineer to insure them a well constructed bridge.

From the writer's experience it is difficult to impress upon the minds of the majority of county commissioners the importance of a properly designed bridge, and the only way to provide safe bridges, in the writer's opinion, is the enactment of stringent uniform laws by the various state legislatures, somewhat after the fashion outlined above.

Recommendations Concerning the Units of Force*

By Edward V. Huntington†

The most casual inspection of our current text-books in Mechanics will convince any reader that the fundamental relations between force, mass and weight, as ordinarily presented, must be thoroughly perplexing to the beginner; and every teacher of elementary mechanics knows well that this is the fact. The comparatively trivial matter of the choice of units occupies much too prominent a place in the ordinary treatment, and "difficulties about units" are too apt to obscure, in the student's mind, much more vital portions of the subject.

What sense, for example, can a student be expected to make of the following statement, taken from a standard treatise which ran through many editions both in German and in English, and is still often cited as an authority?

The mass of a 20-lb. heavy body is 0.62 lb.; and inversely, the weight of a mass of 20 lb. is 644 lb. (Weisbach's "Mechanics.")

Or how can the student be expected to disentangle, without long effort, the various so called "systems of units" which it may or may not be allowable to use in connection with the equation F = ma?

It may be mentioned in passing that precisely this attempt to compel all systems of units to fit the specialized equation F = ma is the source of most of the difficulties in this connection. Without going into the history of the subject, it must suffice here to say that these difficulties are very real ones, which seriously hinder the progress of many intelligent students.

In the writer's opinion, all these difficulties could be entirely removed by adopting the following very simple recommendations, which introduce no new terms or concepts, but serve simply to make a little more precise the modes of expression which are already in current use in the engineering profession.

Throughout the discussion, a force is regarded as anything of the nature of a push or a pull (in whatever direction the push or the pull may be exerted); and the natural instrument for measuring forces is taken to be the spring balance, for we have a right to assume, at least in theory, that the force required to stretch a spring a given amount will always be the same, whenever and wherever the instrument may be used.

The problem then is: What is the unit in which a spring balance should read? That is to say: How shall we define an easily reproducible physical force which will stretch the spring of a standard spring balance to the point marked 1 on the scale?

Recommendation 1. Adopt, as the fundamental unit of force, the standard pound force (1 lb.), or the standard kilogram force (1 kg.); that is, the force required to support a standard pound body, or a standard kilogram body, against gravity, in vacuo, in the standard locality.

Here, by the "standard locality" we mean any locality where g, the observed acceleration of a body falling freely from rest, in vacuo, has the standard value g_s, as adopted by the International Bureau of Weights and Measures (Paris, 1901), namely:

$$g_s = 980.665 \text{ cm. sec.}^2 \text{ or } 32.1740 \text{ ft. sec.}^2.$$

[Note. This value g_s was originally intended to be the value of g at sea-level, 45° latitude; but if subsequent measurements should ever show that the actual value of g at sea-level, 45° latitude, differs somewhat from the number here given, this number would still be retained as the international definition of g_s.]

The adoption of this recommendation would do away at once with all the confusion that necessarily results from the very prevalent habit of defining a "pound force" as the "force required to support a pound body in any locality where one happens to be." Since the force required to support a given body varies in different localities, this latter definition is, strictly speaking, no definition at all; for, to speak of a variable quantity as a unit is in reality a contradiction in terms. It is just as absurd to select the variable force of gravity on a pound body in different localities as a standard of comparison in the measurement of forces, as it would be to select the variable length of an iron bar at different temperatures as a standard of comparison in the measurement of lengths. Every unit of measurement should be a definite and invariable quantity.

As to the choice of the "standard locality," it is obvious that a locality which has already been recognized as the standard, by international agreement, is to be preferred to any of the national capitals, such as London or Paris.

The numerical relation between the standard pound and the standard kilogram, as defined by Act of Congress, is:

1 lb. = 0.45359 kg.; 1 kg. = 2.2046 lb.

Recommendation 2. If any other units of force are introduced for any purpose, wise or unwise, let them be defined explicitly in terms of the standard pound force or the standard kilogram force. For example (to mention both the wise and the unwise):

1 ounce = $\frac{1}{16}$ lb. 1 poundal = $\frac{1}{32.1740}$ lb. 1 stone = 14 lb.

1 gram = $\frac{1}{1000}$ kg. 1 dyne = $\frac{1}{980665}$ kg. 1 ton = 2000 lb.

1 hundredweight (cwt.) = 112 lb. 1 long ton = 2240 lb.

1 "London pound" = $\frac{32.19}{32.1740}$ lb., and 1 "London kilogram" = $\frac{32.19}{32.1740}$ kg. (where 32.19 ft./sec.² is assumed as the value of g in London).

1 "New York pound" = $\frac{32.16}{32.1740}$ lb., and 1 "New York kilogram" = $\frac{32.16}{32.1740}$ kg. (where 32.16 ft./sec.² is assumed as the value of g in New York).

Recommendation 3. Discourage the use of the "poundal" altogether, and do not use the "dyne" except in treating the special class of problems in which all the data are given in C. G. S. units.

(No spring balances graduated to read in "poundals" or "dynes" have ever been placed on the market.)

Discourage also the use of all the so called "local units," such as the "London pound," the "Paris kilogram," etc., as well as all the antiquated units like the "stone" and the "hundredweight."

Recommendation 4. Recognize the distinction between the local weight and the standard weight of a body, as follows:

The local weight of a body, in any specified locality, is the force of gravity on that body in that locality.

The standard weight of a body is the force of gravity on that body in the standard locality (where g has the standard value g_s).

Here, by the "force of gravity" on a body, we mean simply the unseen force which gives the body, when allowed to fall freely from rest, in vacuo, in the given locality, the observed acceleration g. It is equal and opposite to the force required to support the body in that locality.

(Note that this definition of the "force of gravity" on a body is stated in terms of directly observable facts. The ordinary definition: "Force of gravity = the attraction of the earth," is decidedly objectionable, on account of the complications connected with the spheroidal shape of the earth, the influence of the earth's rotation, etc.)

The relation between the local weight, W_l, and the standard weight, W_s, of a body is given by the proportion

$$W_l/g_l = W_s/g_s,$$

where g_l is the local value of g, and g_s the standard value of g.

For example, if the body in question is a standard pound body, then its standard weight, W_s, will be 1 lb.; but its local weight, W_l, will be greater or less than 1 lb., according as the local value of g is greater or less than g_s.

Recommendation 5. Distinguish clearly between the two processes commonly called "weighing." Thus:

(a) "Weighing" a body on a spring balance gives directly the local weight of the body. For, since the body is at rest, the upward force exerted by the spring must be exactly equal to the downward force of gravity. On the other hand:

(b) "Weighing" a body on a beam balance, against a "set of standard weights," gives directly the standard weight of the body. For, a "set of standard weights" is simply a set of metal pieces, each of which is marked with a number which shows its standard weight.

Note that the weight of a body in any case is a force, and should therefore always be expressed, like any other force, in standard pounds or standard kilograms.

Recommendation 6. Recognize the general equation $F \div F_1 = a \div a_1$, or $F \div W = a \div g$, as the fundamental equation of dynamics, and do not use the special equation F = ma (or m = W ÷ g) except in treating the special class of problems in which all the data are given in C. G. S. units.

If this recommendation is adopted, no possible "difficulty about units" can arise, since only forces, not masses, occur in the fundamental equation, and the ratio of two forces will

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always be the same, no matter what unit of force is employed.

Moreover, if this fundamental equation is adopted, it is clear that any special discussion of the "units of mass" becomes entirely superfluous. For example, it is immaterial, for all dynamical purposes, whether a given body is described as "a mass of five pounds," or as having a "standard weight of five pounds." In fact, the standard weight of a body is the simplest and most natural measure of the inertia of the body: for, from the fundamental equation in the form $F \div W_s = a \div g_s$ where g_s is a known constant, we see that if we double or treble W_s, the standard weight of a body, we thereby double or treble the force F required to give the body any specified acceleration a. And so in general: If the standard weight of a body is known, we can at once compute the acceleration which any given force will produce in that body.

(Note that this statement is nothing more than an accurate formulation of the commonest fact of observation concerning moving bodies, namely, that the heavier a body is, the harder it is to set it in motion.)

Thus, by the simple device of introducing the adjective "standard," in speaking of force and weight, we can at once secure complete scientific accuracy of statement without sacrificing any of the convenient usages of common language.

Recommendation 7. If the concept of mass is introduced at all (as distinguished from standard weight), adopt the pound mass and the kilogram mass as the only fundamental units of mass, and discourage altogether the use of such units as the "slugg," the "geepound," or the "wog."

(No set of standard weights marked in "sluggs," "geepounds," or "wogs" has ever been placed on the market.)

Recommendation 8. Whenever there is the slightest danger of confusion between the pound mass and the pound force, or between the kilogram mass and the kilogram force, write out these terms in full. (This confusion will not occur, however, if the preceding recommendations are adopted, since only forces, not masses, occur in the fundamental equation.)

If these recommendations could be generally adopted by text-book writers on elementary mechanics, the labors of both teacher and student would be materially simplified. For a more detailed presentation of the arguments in favor of these recommendations, the reader is referred to the complete article already mentioned, of which this paper is an abstract.

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The Treatment of Sewage Discharged into Tidal Waters

Anomalies in sewage-treatment requirements for various municipalities on the River Thames, England, afforded a text for a discussion of "The Treatment of Sewage Discharged into Tidal Waters," by H. C. H. Shenton, before the British Association of Managers of Sewage Disposal Works on June 21. The meeting was held at Erith, a town of about 28,000 population in 1911, which led Mr. Shenton to state:

Here we see a town situated immediately below and even within sight of the Barking and Crossness outfalls which discharge the whole of the London sewage, and while this immense volume of Metropolitan sewage is merely treated by chemical precipitation, the relatively small volume discharged by Erith is treated in such a manner that it is comparable with the best effluents discharged higher up the river above the intakes of the Metropolitan Water Board. It should be observed that the apparent inconsistency of making Erith purify its sewage to this high degree is present elsewhere on the River Thames.

It seems as if the authorities had only one standard which is applied at all places regardless of conditions. The same quality of effluent is demanded at Staines, above the intakes of the Metropolitan Water Board, at Kingston, below the intakes but above locks, and at Erith in tidal water which is already grossly polluted.

In demanding this high degree of purification the authorities in the past were probably of opinion that the quality of the sewage effluent should have no reference to the quality of the water into which it was discharged, for if the sewage of Erith had only to be purified to such a degrees as to make it of the same quality as the water in the River Thames at the point of discharge, it is certain that the greater part of the works might have been omitted.

After referring to the reports of the Royal Commission on Sewage Disposal and citing the American opinions of George W. Fuller, George C. Whipple and the Metropolitan Sewerage Commission of New York City, Mr. Shenton concludes his paper as follows:

It is clear that the treatment of sewage discharged into tidal waters must vary from the discharge of crude sewage or tank effluent at places where it can do no possible harm, to the most complete system of tanks and filters, sterilization being generally held in the background as a final possibility. These facts show the growing importance of scientific consideration of each case where sewage has to be discharged into tidal waters. The chemist, bacteriologist, pathologist and engineer must each have a great deal of work to do in order to ascertain the exact degree to which it is necessary to purify the sewage at any given place, and such scientific attention will often result in the saving of a very large sum of money on constructional works and maintenance; but where this skilled attention is not given in the first place, it will generally be necessary for the authority to put in works of an elaborate nature which may sometimes be altogether in excess of the requirements.

It is not to be imagined that sewage can often be discharged into tidal waters without any purification, but the degree of purification required before the effluent is discharged must vary considerably, and its attainment will depend not only upon the design of the works, but upon the management which they receive. Further, that the judicious improvement and better working of existing systems may be attained by the efforts of the Manager is clear, and of this fact the works at Erith are a striking example.

The Erith Sewage-Disposal Works

In view of Mr. Shenton's references to the Erith works, the following description of them, from the London "Surveyor," of July 4, 1913, is of interest:

The Erith sewage-disposal works were opened in September, 1898, as part of a sewage scheme undertaken by the council at a cost of £80,000, £20,000 being devoted to the construction of the sewage-disposal works, the remaining £60,000 expended in sewerage works throughout the district. The scheme was prepared by George Chatterton, and the works constructed by G. Osenton. The population at the time of opening was (approximately) 15,000.

The works, with an area of two acres, were designed for the chemical treatment of sewage, cream of lime and aluminoferric being used as a precipitant, the resultant sludge from the three precipitation tanks being pressed into cakes by two presses.

The council hoped that the proceeds obtained by the sale of the pressed sludge would materially assist in paying for the upkeep of the works; but farmers who tried this sludge-cake as a manure did not care for it, possibly on account of the greasy nature of the sludge due to the presence of the aluminoferric.

After three years' working on this process it was discontinued, the accumulation of pressed sludge during that time filling up all the available space at the works. The rams and presses, etc., were eventually sold.

The Thames Conservancy being dissatisfied with the sewage effluent discharging into the river Thames from the precipitation tanks, the council decided to adopt the bacteriological treatment of sewage by converting the precipitation tanks into open septic tanks, and constructing two contact beds, which were excavated in the clay, and later four larger contact beds were constructed with concrete sides and bottoms.

Subsequently the council decided to construct continuous aërating filters with Adam's patent revolving distributors. The effluent channel was constructed at sufficient height to allow the effluent to discharge into the river at all states of the tide, with the exception of extraordinary high tides, when the effluent would head up in the channel and overflow into the storage tank. This effluent channel is about 440 ft. in length and 4½ ft. wide, and contains three catch-pits for arresting humus passing from the filters.

The council, finding that the effluent from the continuous aërating filter satisfied the Thames Conservancy, have from time to time constructed four more filters of a similar type, and the ground is prepared for another; they have also acquired 1½ acres of land on the west side of the works for further extension.

Seepage Losses from Earth Canals

By E. A. Moritz*

The articles that have appeared recently in technical papers on the subject of seepage losses from canals are an indication that an increasing amount of scientific thought is being given to this very important subject, which becomes ever more important as the available water-supply diminishes and becomes more valuable.

We commonly speak of these losses as "seepage and evaporation losses," but the loss from evaporation is such a comparatively insignificant item that it does not require serious consideration and should not be put on the same plane with seepage or percolation. There are other losses which probably amount to much more than the evaporation in most canal systems, such as leakage through banks and around and through structures, and illegal abstraction. Leakage and illegal abstraction are not susceptible of calculation, but in a well ordered system these should not be important items, and in the present discussion only the nonpreventable losses from seepage or percolation through the bed and banks of the channel are considered.

Terms in Which Seepage Losses Should Be Stated —Various units have been used for expressing seepage losses, among which are: Cubic feet per second per acre of wetted area; per cent. of flow per mile; feet depth over the water surface per unit of time; per cent. of total diversion; and feet depth over the wetted area per day. Until recent years the per cent. of flow unit was the most popular—no doubt because of its simplicity of manipulation—but the irrational basis of this unit is now generally conceded and losses are commonly expressed in feet depth over the wetted area in a given time.

Fig. 1. Diagram Illustrating Rate of Percolation in Earth Canals

(For convenience, length of water slope has been taken as twice the water depth.)

It is a generally accepted fact that the velocity of ground water varies directly as the pressure or hydraulic gradient and it has, therefore, been assumed that the rate of percolation of water through canal banks varies directly as the depth of water in the canal—assuming, of course, that the bottom of the canal is higher than the natural ground-water level of the adjacent land. Granting this to be true, the rate of percolation may be illustrated by the accompanying diagram (Fig. 1) in which the lengths of the arrows represent the velocities of flow of seepage water at the respective points.

Now if the rate of loss is expressed in per cent. of flow per unit of length or in per cent. of diversion, this rate will vary with the depth of water in the canal, but if the loss is expressed in terms of the depth over the wetted perimeter in a given time—say 24 hours—the effect of depth of water is to a large extent automatically eliminated. This may be demonstrated as follows:

Let

c = average depth of loss over wetted perimeter,

c_1 = maximum depth of loss, occurring at the bottom of the canal and decreasing uniformly to zero at the water surface.

Now assuming any fixed quantity, Q, velocity, V and area $A = \frac{Q}{V}$ it is only necessary to show that the sum of the areas E, F and G will be practically equal to the average area $c\,(b + 4d)$ for any ordinary ratio of b to d, that is:

$$c\,(b + 4\,d) = c_1\,(b + 2\,d)\ or\ \frac{c}{c_1} = a\ constant \qquad (1)$$

First assume a ratio of b to d of 2 to 1 as the minimum, then:

$$c\,(6d) = c_1\,(4d)$$

or

$$\frac{c}{c_1} = \frac{4\,d}{6\,d} = \frac{2}{3} = 0.667 \qquad (2)$$

Now assume $b/d = 8$ to 1 as the other extreme, then $c\,(8d + 4d) = 12\,cd$ and $c_1\,(8d + 2d) = 10\,c_1 d$ and

$$\frac{c}{c_1} = \frac{10\,d}{12\,d} = \frac{5}{6} = 0.833$$

The deviation from the assumption made above, namely, $\frac{c}{c_1}$ = a constant, then is $\frac{0.833 - 0.667}{0.833\ or\ 0.677} = 20\%$ or 25%, depending on whether c is obtained from seepage measurements from canals having the larger or the smaller ratio of b to d. Actually, the measurements of the value of c are obtained from canals having water sections averaging between these two extremes so that the actual error due to the assumption that $\frac{c}{c_1}$ = a constant will have a maximum value of about 10% for extreme canal sections and much less than this for the average run of section. This is closer than the average depth of seepage loss c can be estimated for any given material, hence, it is clear that it is not essential that the depth of water in the canal be taken into account in calculating seepage losses. This is, moreover, very fortunate, as there are no experimental data to aid us in establishing the necessary constant.

Data on Seepage Losses—Much labor and expense are required to obtain accurate data on the value of c for different materials and on this account, and also no doubt because few canal systems are provided with adequate facilities for measuring the flow, few observations have been made and little has appeared in the technical press on the subject except very general observations. The subject is, therefore, brought up again in this brief form in the hope that other engineers will add such data as they may have accumulated so that the aggregate fund of available information may reach something like the proportions which the importance of the subject warrants.

The following figures represent the average results obtained from observations on several hundred miles of canals on eight different irrigation projects of the U. S. Reclamation Service and give a fair idea of what may be expected in the way of seepage losses from canals in the different materials represented. The losses are ex-

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pressed in terms of the depth in feet lost in 24 hours through the wetted perimeter of the canal prism:

Kind of material	No. of observations	Loss
Cement gravel and hardpan with sandy loam	3	0.34
Clay and clay loam	5	0.41
Sandy loam	4	0.66
Volcanic ash	3	0.68
Volcanic ash with some sand	5	0.98
Sand and volcanic ash or clay	8	1.20
Sandy soil with some rock	3	1.68
Sandy and gravelly soil	8	2.20

These are generally results from canals that have been in operation from three to six years. There is usually a very noticeable reduction in seepage losses with continued use, especially if the water carries fine silt, and there are instances where the most porous gravel formation has been made practically watertight by a coating of silt or puddle. In designing a canal, it is probably unsafe to figure on a smaller loss than 0.5 ft. over the wetted area in 24 hours in even the most impervious material and after a loss of over 2 to 2.5 ft. is reached, the question of lining the canals will generally require very serious consideration from the point of view of value of the water and damage to adjoining lands from waterlogging. The limits within which seepage losses should be considered may, therefore, be generally defined as 0.5 ft. and 2.5 ft. per day over the wetted area of canal, for the minimum and maximum respectively.

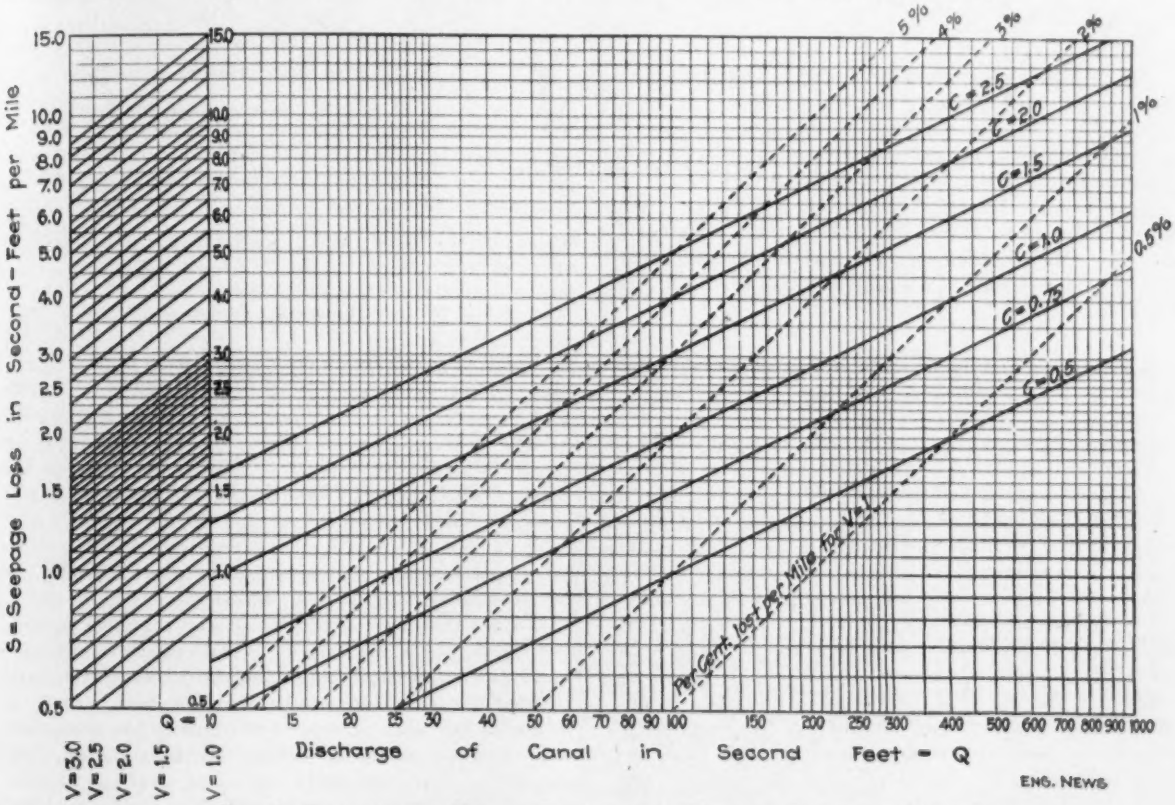

FIG. 2. DIAGRAM FOR USE IN CALCULATING SEEPAGE LOSSES IN CANALS

$$\left(\text{Equation } s = 0.2\, c = \frac{Q^{\frac{1}{2}}}{V^{\frac{1}{2}}}\right)$$

DIAGRAM FOR ESTIMATING SEEPAGE LOSSES—As an aid in the calculation of seepage losses, the diagram, Fig. 2, has been used with satisfactory results. The assumptions on which this diagram is based, as well as its construction, will be explained in detail in order that its limitations and manner of use may be readily grasped. Side slopes of 1½ to 1 are most generally used for irrigation canals, and since the calculated seepage losses vary only slightly for ordinary deviations from this slope, it is assumed constant at this value.

Let

V = velocity of water in feet per second;
Q = discharge of canal in second feet;
b = bottom width of canal in feet;
d = depth of water in canal in feet;
p = length of wetted perimeter in feet;
a = area of water section in square feet;
n = ratio of bottom width to depth = b/d;
A = total area irrigated in acres;
K = area irrigated per second foot in acres;
c = depth of water in feet lost over wetted area in 24 hours;
s = loss in second feet per mile of canal;

Then

$$a = bd + 1.5\, d^2 \quad n = b + 3.6\, d = \frac{a}{d} + 2.1\, d$$

Total loss in 24 hours per mile of canal = $5280\ p\ c$ cubic feet.

Total loss in cubic feet per second $\frac{5280\ pc}{86{,}400} = 0.0611\ pc = s$.

$$Q = \frac{A}{K}, \quad a = \frac{A}{KV}$$

Since $a = bd + 1.5\,d^2$ and $b = nd$,

$$a = nd^2 + 1.5\,d^2 = (n + 1.5)\,d^2 \text{ and } d = \left(\frac{a}{n + 1.5}\right)^{\frac{1}{2}}$$

$$p = \frac{a}{d} + 2.1\,d = \frac{a}{\left(\frac{a}{n + 1.5}\right)^{\frac{1}{2}}} + 2.1\left(\frac{a}{n + 1.5}\right)^{\frac{1}{2}}$$

$$= a^{\frac{1}{2}}\,(n + 1.5)^{\frac{1}{2}} + 2.1\left(\frac{a}{n + 1.5}\right)^{\frac{1}{2}}$$

$$= a^{\frac{1}{2}}\left[(n + 1.5)^{\frac{1}{2}} + \frac{2.1}{(n + 1.5)^{\frac{1}{2}}}\right]$$

Since

$$a = \frac{A}{KV}, \quad a^{\frac{1}{2}} = \frac{A^{\frac{1}{2}}}{K^{\frac{1}{2}}V^{\frac{1}{2}}}$$

Therefore

$$p = \frac{A^{\frac{1}{2}}}{K^{\frac{1}{2}}V^{\frac{1}{2}}}(n + 1.5)^{\frac{1}{2}} + \frac{2.1}{(n + 1.5)^{\frac{1}{2}}}$$

and since $s = 0.0611\,pc$,

$$s = 0.0611\,c\left[(n + 1.5)^{\frac{1}{2}} + \frac{2.1}{(n + 1.5)^{\frac{1}{2}}}\right]\frac{A^{\frac{1}{2}}}{K^{\frac{1}{2}}V^{\frac{1}{2}}}$$

The value of the factor in brackets is 3.24 for $n = 4$ and 2.91 for $n = 1$ showing such a small variation in value for a large difference in n that this variation may be neglected and n given a constant average value of 3.24. Then

$$s = 0.0611 \times 3.24\,c\,\frac{A^{\frac{1}{2}}}{K^{\frac{1}{2}}V^{\frac{1}{2}}} = 0.2\,c\,\frac{A^{\frac{1}{2}}}{K^{\frac{1}{2}}V^{\frac{1}{2}}}$$

or

$$s = 0.2\,c\,\frac{Q^{\frac{1}{2}}}{V^{\frac{1}{2}}} = \textit{seepage loss in cubic feet per second per mile of canal}$$

The effect of variations in the dimensions of the canal section having been demonstrated to be negligible within practical limits, the final equation for seepage loss involves only three variables, namely, the canal discharge Q, the mean velocity of flow V and the experimental coefficient c. If reasonably good data are available as to the value of c this equation will give with practical accuracy the probable loss from seepage in canals of any size or shape generally used in practice. The number of calculations necessary to determine the total losses accurately is obviously dependent upon the rapidity with which the canal is decreasing in capacity and also upon the accuracy that is desired. This is entirely a question of individual judgment, but it must be remembered that whether water is being drawn from the canal or not the seepage losses continue and the capacity must be reduced at intervals by the amount of these losses.

The equation $s = 0.2\,c\,\frac{Q^{\frac{1}{2}}}{V^{\frac{1}{2}}}$ has been plotted on the diagram, Fig. 2, with values of Q as abscissae and values of s as ordinates, the diagonal lines corresponding to different values of c. To care for variation in the velocity of flow an auxiliary scale has been constructed at the left. The scales are all logarithmic, but the diagram is very easily constructed by transferring the scales from a common slide rule. The auxiliary scale for different values of V requires a brief explanation: The base scale at the left is for a velocity of 1 ft. per sec., and its use may be demonstrated by the following example:

Assume $Q = 100$; $c = 1.5$; and $V = 1$.

Enter the diagram at the bottom and follow up the vertical line marked $Q = 100$ to its intersection with the diagonal line marked $c = 1.5$, thence horizontally to the base scale at the left, reading $s = 3$ cu.ft. per sec. per mile. Now, if the velocity is 2 ft. per sec. instead of one, the other factors remaining the same, we do not stop at the base scale for $V = 1$, but continue down the diagonal line to the left to its intersection with the vertical line marked $V = 2$ reading $s = 2.12$, or, if the velocity is 3 ft. per sec., we continue down the diagonal line to its intersection with the vertical line for $V = 3$ and read $s = 1.72$. The advantage of using as high velocities as possible is forcibly illustrated by these figures. The manner of constructing the auxiliary scale is as follows: Draw the vertical line marked $V = 3$ any convenient distance, say 1½ in. to the left of the $V = 1$ line; now since s varies inversely as the ½ power of V the ratio of loss at 1 ft. per sec. to the loss at 3 ft. per sec. is $\frac{\sqrt{3}}{\sqrt{1}} = \sqrt{3}$ and if s_1 is the loss at 1 ft. velocity and s_3 the loss at 3 ft. velocity $s_1 = s_3\,\sqrt{3}$ or $s_3 = \frac{s_1}{\sqrt{3}}$; therefore, for $V = 3$, all the figures on the scale for $V = 1$ must be divided by $\sqrt{3}$ and to accomplish this graphically we draw the diagonal lines from any value s on the $V = 1$ line to the value $\frac{s}{\sqrt{3}}$ on the $V = 3$ line. To locate the intermediate lines for $V = 2.5$, 2 and 1½ we mark the distances out in the ratio of $\frac{\log 2.5}{\log 3}$, $\frac{\log 2}{\log 3}$ and $\frac{\log 1.5}{\log 3}$ respectively, that is, if the $V = 3$ line is 1½ in. from the $V = 1$ line, the $V = 2$ line will be $1\frac{1}{2} \times \frac{\log 2}{\log 3} = 1\frac{1}{2} \times \frac{.3010}{.4771} = 0.95$ in. from the $V = 1$ line, etc. Lines for intermediate or higher values of V may be located in a similar manner.

Irrationality of Percentage Unit—The dashed lines shown in the diagram are not necessary for the determination of seepage losses, but have been inserted to illustrate the magnitude of the error involved in stating the loss in per cent. of flow of the canal. For example: take two canals in similar material having a seepage factor c of 1. The capacity of one canal is 100 cu.ft. per sec. and that of the other is 500 cu.ft. per sec. The canals, being in similar material, are assumed to have the same percentage of loss. Entering the diagram at $Q = 100$, we note that the loss for $c = 1$ is 2% per mile and for $Q = 500$ and $c = 1$ we note the loss to be 0.9 per cent. per mile. It is a generally accepted fact that the rate of loss decreases as the capacity of a canal increases, but the rate of such variations has not been understood and more often than not a single percentage has been assumed for all sizes of canals. The rate of change in the per cent. of loss from canals of varying capacity as well as the absolute loss is, moreover, dependent upon the velocity of flow as well as the capacity and a different gradation of percentages would have to be allowed for the different velocities if the percentage unit were used. This unit is, therefore, untenable and should be entirely abandoned except perhaps for very rough estimates. This has fortunately already been done by most engineers, but there are no doubt some who still adhere to the old

method and for those a perusal of the evidence given in the above diagram will be especially interesting.

Effect of Velocity on Seepage Losses—The great reduction of seepage losses that results from an increase in the velocity of flow has been noted above. In designing canal systems, the matter of proper velocities is generally a vexatious question, and is usually based on a consideration of the available grade and scouring and silting values of the velocity. If plenty of fall is available, the highest possible velocity that will not scour the banks will generally be used, and if all possible saving is to be made in grade, a velocity will be used of such magnitude only that excessive depositing of silt will not occur. The increased loss of valuable water from seepage, due to low velocities, often followed by serious damage to adjoining lands, is not usually given the consideration that it deserves. The magnitude of the effect of low velocity is made apparent by the figures above, and by other comparative figures that may be taken from the diagram. If, as noted, for a canal of 100 cu.ft. per sec. capacity an increase in velocity—other conditions remaining the same—from 1 to 3 ft. per sec. causes a reduction in seepage loss from 3 to 1.72 cu.ft. per sec. per mile, the effect of the velocity certainly warrants careful and serious consideration from this standpoint.

Puddling or Blanketing Canals to Reduce Seepage—From a physical standpoint, concrete lining is the best means of stopping seepage, but from a financial standpoint, other methods may be better. Where suitable material is available a great saving may often be effected by lining the bed and banks with an earth puddle. Volcanic ash is excellent for this purpose. Two methods have been used: In the one a blanket of tamped material from 6 to 12 in. thick is placed over the wetted area before water is turned into the canal; and in the other puddle material is thrown into the water, which is either quiet or moving slowly, and is worked into the bed and banks principally by men wading with rubber boots. In very porous material it may be necessary to plug up some of the worst places with sacks filled with the puddling material; the first mentioned can only be used satisfactorily, without excessive expense, when the blanketing material is moist, or water readily available for moistening it. The latter condition does not generally obtain on the construction of irrigation canals.

The writer has had some experience with each of the above methods. In one case when the puddle method was used, the canal was designed for a capacity of 100 cu.ft. per sec., a bottom width of 6 ft., and a depth of 3.7 ft.; with 1½ to 1 side slopes. It was made comparatively narrow, on account of being in a deep cut. The material in which it was built was a very porous gravel—the conditions in fact were so bad that the landowners in the neighborhood threatened to get out an injunction to have the work stopped, or to force a concrete lining to be used, for fear that the leakage would be so great as to swamp their lands; and an engineer of good judgment, who had had much experience with canals in the same material offered to drink all the water that reached the end of the canal, which was 1½ miles in length. The rapidity with which the water disappeared when the first head was turned in justified the fears that had been expressed, but after two weeks of constant labor, day and night, and the expenditure of about $2000—equivalent to about 10c. per sq.yd., the canal was made practically water-tight, as shown by measurements over sharp-crested weirs at either end. Concrete lining 4 in. thick would probably have cost not less than $25,000.

A puddle lining cannot be made in a day; it requires much patience and constant attention during the first season or two to produce permanent results; but when the saving of expense is an object, as it is on most irrigation enterprises, the saving effected over concrete, or other similar forms of lining, may be very considerable in the end. There are cases, of course, where a puddle lining would not be practicable, and in other cases where plenty of fall is available a concrete lining can often be used to greater advantage by using a small canal section and high velocities, which would not be possible in a puddled canal.

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Domestic and Foreign Records of Fire Loss for 1912

The Committee on Statistics of the National Board of Fire Underwriters has just submitted its annual report on the fire losses in the United States and some foreign countries for the calendar year 1912. From an abstract of the report appearing in the *Quarterly* of the National Fire Protection Association, July, 1913, the following table of losses in cities over 20,000 population has been taken.

COMPARATIVE FIRE LOSSES OF DIFFERENT COUNTRIES IN 1912

[Cities over 20,000 population]

	No. of cities reporting loss	Population	Per capita loss
United States	300	32,326,633	$2.55*
England	12	7,164,849	0.54
France	6	4,425,696	0.84
Germany	9	2,650,575	0.20
Ireland	2	699,802	0.57
Scotland	2	485,091	0.49
Italy	3	282,082	0.90
Russia	2	3,485,583	0.84
Austria	4	2,658,978	0.30
Canada	5	957,372	2.88
Argentina	1	1,428,042	3.58

* Estimated per capita loss for whole country = $2.16.

It will be noted that the United States continues to lead the old world in the per capita loss; it should also be noted that Canada's large cities lead those of this country by 12% and that one city in Argentina, presumably Buenos Aires, has the high per capita record, $3.58. It is thus evident that excessive fire losses are peculiar to the newer countries, where construction is not so permanent nor the necessity of guarding wealth so apparent.

Figures such as these are constantly being used to illustrate the backwardness of the United States in fire-protection work. In general this is perfectly legitimate but in presenting the data attention should be called to the great disparity in values between the United States and Europe, which tends to reduce considerably the ratio between the respective fire losses. Labor and most materials are cheaper across the ocean; consequently a $5000 loss there is greater than a $5000 loss here. While there is no doubt that fire destroys altogether too much property here the relative freedom from fire loss is not precisely shown by the per capita losses shown in the above table.

An almost unbelievable record in the report is that from Dresden, Germany, a city of 561,000 people, with a total fire loss in 1912 of $16,800 or only *3c. per capita.*

The Truth about the Culebra Cut Slides, Panama Canal

By A. S. Zinn*

The method suggested for preventing earth slides by constructing underground retaining walls as described in the article, "A Grouting Process for Preventing Slides in the Panama Canal," by George S. Rice, in Engineering News of June 5, 1913, is interesting, but the plan would not be practical with the slides along Culebra cut, Panama Canal. In fact, the scheme would be impossible to carry out, and I feel sure that Mr. Rice would come to the same conclusion if he were to come here and make a study of the actual conditions.

Fig. 1 is the cross-section of the slide at Sta. 1743, as printed in 1912 Annual Report of the Isthmian Canal Commission and used by Mr. Rice in the paper read before the Western Society of Engineers, a portion of which is reprinted in the issue of Engineering News previously referred to. I have shown on the cross-section by hatched lines the quantity of material removed between July 1, 1912, and June 1, 1913. This shows the canal prism almost completed at this point. I have shown thereon Mr. Rice's plan for the underground retaining wall with tie rods down to mean sea level at *C*, *D*, *E* and *F*, and have also shown results of borings to 40 ft. below mean sea level, as made under the direction of our first Chief Engineer, John F. Wallace, in 1904 and 1905. There is practically the same soft material throughout the extent of the excavation on both sides of the center line of the canal at this point. Although classified as soft rock, it was necessary to blast all of it except the contents of slide itself before the material could be handled by steamshovels

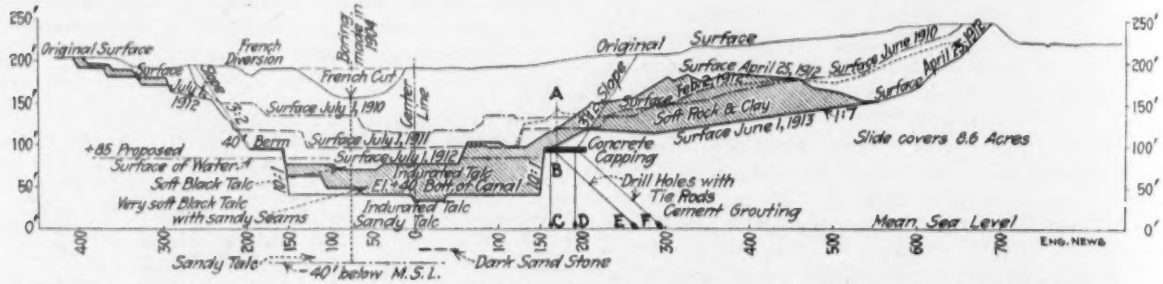

Fig. 1. Cross-Section of Culebra Cut, Panama Canal, at Sta. 1743, Showing a Proposed Scheme for Preventing Slides

Had holes been drilled for tie rods as shown in Fig. 1, no hard rock would have been reached at sea level as assumed by Mr. Rice. The borings made show that there is a sandy talc 40 ft. below sea level. Borings were taken in 1904 and 1905 throughout the entire length of Culebra cut to 40 ft. below mean sea level, and not one of them would indicate that a hard stratum would be reached even at 40 ft. below sea level.

It is impossible for a geologist or anyone else to say positively just where a slide is going to develop, and after it has started to move it would be absolutely impossible to drill the holes, for the moving mass would not only soon close the holes but would push the drills out into the cut, or cover them over with sliding material. Most of the theoretical slopes along where the slides developed were furnished by the first Board of Consulting Engineers and are as shown on the cross-section at Sta. 1743 (Fig. 1), that is, 10 on 1 from canal bottom at an elevation of 40 ft. up to +95 ft., then a 40-ft. berm, and from there up on a slope of 3 on 2.

All material excavated beyond the theoretical slope lines has been considred in our estimates as due to slides, so that up to the present time a little over 22,000,000 cu.yd. has been excavated from slides. About 82%, or 18,000,000 cu.yd., was taken out before excavating below the berm at an elevation of +95 ft., showing that only about 18%, or 4,000,000 cu.yd. has been taken out below the berm at +95.

Culebra cut extends from the south end of the Pedro Miguel locks to the Chagres River, a distance of nine miles. The total length of slides along the east bank is 20,800 ft., and the total length of slides along the west bank is 10,400 ft., or a total length of all slides of 31,200 ft., or about six miles—one-third the total length of the two sides of Culebra cut. The total area of slides at the present time (all material outside of theoretical slopes) is 251 acres.

The excavation of the slides has been a simple matter. Some of the highest steamshovel records have been made at the toes of slides, and as the material requires very little blasting the average cost per cu.yd. on all slides has been close to 45c. per cu.yd., or a total cost of $9,900,000.

Assuming that Mr. Rice's retaining-wall plan is practicable with the top of the concrete capping at an elevation +95, and also assuming that it would prevent any movement of slides below +95, we would then have saved the excavation of 4,000,000 cu.yd.

The cost of the drilling and blasting, the tie rods, and the concrete for underground retaining wall would not be less than $250 per lin.ft. of face of slide, or a total cost for all slides of $7,800,000. This sum would have been expended to save 4,000,000 cu.yd. of excavation, or at a cost of $1.95 per cu.yd. or $6,000,000 more than it has actually cost the Isthmian Canal Commission. This shows that the cost alone would have made the plan prohibitive.

Referring again to the cross-section at Sta. 1743, Fig. 1, it will be seen that the excavation has been made to the bottom of the canal, on a slope of 10 on 1 below the 40-ft. berm at elevation +95 without the slide breaking below +95; and that slide is now dead. Therefore, if the underground retaining wall had been constructed here it would have been a useless waste of time and money.

The slide at Sta. 1743 is the smallest of the big slides,

*Resident Engineer, Central Division, Panama Canal, Empire, C. Z.

covering an area of about 9 acres and requiring 700,000 cu.yd. of excavation to get the best of it. The largest slide is on the west side of the canal at Culebra, covering an area of about 70 acres, and so far over 8,000,000 cu. yd. have been excavated from it; and 500 lin.ft. of the face is still moving slowly toward the canal prism.

Counting all the slides containing over 10,000 cu.yd., there are in all 31; 22 of them along the east bank and 9 along the west bank. Out of the 22 slides along the east bank only 4 remain active and out of the 9 slides along the west bank only 2 remain active, so that out of a total of 31 slides only 6 remain active at the present time.

The 25 slides now dead are along the completed prism and will probably never give any more trouble. The total excavation from the Central division, which extends from the Pedro Miguel locks to the Gatun locks, a distance of 31.7 miles, to date, is 107,000,000 cu.yd. About 94,500,000 cu.yd. of this have been excavated from the nine miles of Culebra cut. About 95% of the remaining excavation extends over a distance of less than two miles at the Continental Divide opposite Culebra, so that over seven miles of Culebra cut is practically completed. The active slides are along the two miles referred to above and it will require approximately 5,000,000 cu.yd. of excavation to kill them. We are still (July 1) operating 35 steam shovels with an output of over 900,000 cu.yd. per month, so it is only a question of a few more months of digging when Culebra cut will be completed.

As stated above no one can tell with any degree of accuracy when or where a slide will occur, and it is not necessary for one to be a geologist or a civil engineer to see at once the cause of our slides after they have moved into the cut. Most of them show that the whole mass was resting on a smooth surface sloping toward the cut on slopes averaging 2 on 3 in most of the rock slides, an average of 1 on 6 in the earth slides and an average of 1 on 3 where there is a combination of rock and clay. As soon as the material was removed from the toe of the slopes the whole mass started to move toward the cut.

Most all of the slides are at points where there are structural breaks and fault zones, yet the one that has given more trouble than any other is Cucaracha slide, on the east side of the cut just south of Gold Hill and is called by the Commission Geologist a "gravity slide." This slide covers an area of about 48 acres. It gave a great deal of trouble to the French as early as 1887, and the first serious trouble to the Americans was in October, 1907, when it moved entirely across the cut and blocked train movements for seven days.

The moving material in this Cucaracha slide was almost entirely saturated clay. The daily movement at that time (October, 1907) is shown by the accompanying sketch (Fig. 2). It continued giving trouble up to May, 1911, when it ceased moving and remained quiet up to the first part of December, 1912. On Jan. 16, 1913, it moved so rapidly that it caught several dirt cars and continued to move until it covered the entire width of the 300-ft. channel; and it has caused a great deal of trouble ever since. Now it looks as though Cucaracha slide will be the last material excavated to complete Culebra cut.

In the Isthmian Canal Commission report of 1912, p. 210, the Commission Geologist in referring to this slide states that "the end of its activity is now well in sight, however, because the loose surface stone and clay have mostly slid off, exposing several large dikes and flows of basalt which will successfully maintain in place most of the remaining material."

I make the above quotation to prove that even *a good geologist* cannot say with any degree of accuracy just when or where a slide may occur; and so far no better remedy has been suggested than the remedy of taking the weight off the top; and this remedy has been applied at every place where practicable from the very beginning, or before the Geologist came to the Isthmus.

In February, 1909, the Board of Consulting Engineers with a number of the most prominent members of the American Society of Civil Engineers walked all over the slides at that time, and it was their unanimous opinion that they could not see any better method of handling the slides than was in operation at that time. If the first

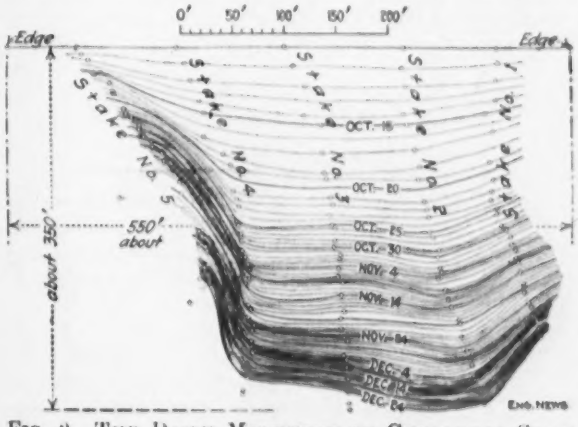

Fig. 2. The Daily Movement of Cucaracha Slide, October, 1907

Board of Consulting Engineers had given us a slope of 1 on 2 above the 40-ft. berm at an elevation +95 instead of 3 on 2, nine-tenths of the slides would never have developed; yet when we estimate the extra amount of excavation it would have required, the 3 on 2 slope has proved to be the economical slope, as 12 miles of the slopes are standing in good condition and will give very little trouble in the future.

To those who have watched big excavation jobs for many years, nothing has happened along the Culebra cut but what could have been expected to happen when one considers that it is the largest excavation job ever attempted anywhere in the world. Many of the slides along the big railroad cuts in different parts of the United States have developed in exactly the same way and no doubt had the excavation been carried to 200 or 300 ft. or more deep as at Culebra the slides would have covered as much area and would have given just as much trouble as they have there.

The most encouraging thing about the slides is the fact that it has been demonstrated that it is only a question of time and continued digging when the last one will reach the angle of repose and will slide no more. Moreover, the appearance of the deep cut has been improved by the slides in making a flatter slope, and thereby eliminating chances for trouble after the water is

in the prism. The heavy blasting has practically caused all loose material along the banks to slide into the cut, so when the blasting is completed and the water turned in there will be very little excavation for maintenance.

As Resident Engineer of this division for about seven years, I have seen every slide develop and have watched the progress of excavation. It has been very discouraging at times to all of us on the job to see a big slide come in when we considered the cut about completed at that point. The slide covering the tracks with rocks and mud, closing the drainage channel in center of canal and occasionally having a steam shovel buried and the bottom of the cut bulged up 10 to 20 ft.—twisting tracks out of shape and interfering with train movements—has naturally been slightly discouraging. Along with it generally comes a heavy rain with a temperature at 96°; yet the obstructions were removed and the damage repaired as quickly as possible even under these most unfavorable conditions. If it had not been for the loyalty, energy and good judgment of every American on the job the completion of Culebra cut would not now be so near the end.

Sliding Ground in Culebra Cut

By Donald F. MacDonald†

The methods of preventing earth and rock slides, proposed by Geo. S. Rice in Engineering News of June 5, 1913, while they may be very applicable in many cases, could not be advantageously applied to the Culebra cut slides, for the following reasons:

(1) The weak crumbly rock in which the slides occur extends several hundred feet below the bottom of the cut. There would therefore be no chance, except possibly at very great cost, of grouting steel rods into a foundation so that they would hold against heavy earth pressure, such as those that might be exerted by one quarter to a million cubic yards of material.

(2) The large slides are not the mere slipping of relatively loose material off a relatively solid base slope, but include a slow movement or deformation of the base slope itself, to depths well below the bottom of the excavation. This often involves a "swelling" upward of the rock floor of the cut. Such movements would greatly complicate and add to the cost of retaining walls.

(3) Slides containing more than a million cubic yards have occurred in the past. A system of retaining walls that would not yield to the earth pressures of such a mass would be a most costly affair, especially as there is no solid rock to make a strong natural foundation for such a system.

(4) There have been large slides in Culebra cut in the past and there are some more yet to come, but these really do not present a problem. Just as soon as the slopes of the cut are made flat enough there will be no more slides. This work of flattening the slopes is now well under way.

(5) The chief reason why there were and are slides in Culebra cut is because the slopes of it were made too steep for the weak and crumbly character of the rocks involved. There are many relatively steep-sided valleys in the Canal Zone. These have been excavated by streams in the same kind of material through which the cut is made. When the slopes of the cut shall approach the natural slopes of these stream valleys, the sliding period will certainly be closed.

(6) The cost of terracing back the upper slopes and making the walls of the excavation slide and deformation proof, will be much less, and will certainly be more effective, than any system of retaining walls could possibly be, under the geological conditions that govern the case.

[As the reader will see that this article substantiates the foregoing one by Mr. Zinn. In justice to Mr. Rice it should be stated that the title of the article in Engineering News was not the title of Mr. Rice's paper as read in full before the Western Society of Engineers. Mr. Rice suggested the method as general and not as particularly applicable to the Panama Canal slides; and he has since received many letters from prominent engineers commending the plan in general.

The articles above are of particular interest in correcting much popular misinformation about the Culebra cut slides, which the reader will see have caused more consternation to the daily newspaper editors than to the engineers in charge of the work.—Ed.]

Bringing an Old Water-Works Valuation up to Date

By William E. Butt*

Let us suppose that a valuation of a certain water-works was made in 1909. It is now 1913. If it was originally $300,000, it may now be $20,000 more or less, at the present time, taking for granted that in each case the appraisal is upon a cost basis only. Cast-iron pipe, lead, hydrants, brick, etc., have very likely changed materially in price. Pipe, valves and pumping equipment may have gone down, while labor and brick have gone up in price, or remain stationary. Are the figures of the old appraisal worth anything in arriving at the valuation in the present year?

Since cast-iron pipe is the chief item in construction of a water-works, it might be suggested that the valuation would change as the price of this item rises or falls. But cast iron, as is well known, fluctuates much more violently than the several other minor elements that enter into the construction, and can hardly be a fair criterion. Again, it might be supposed that the valuation would change with the prices for All Commodities—such as the Index Numbers compiled by the Department of Labor, by Bradstreet, and others.

Table I gives two series of *relative* numbers for the prices of cast-iron pipe and of All Commodities, on the basis of 100, which describe accurately the fluctuation of these prices over the given period.

The writer happens to be in possession of price data for the twelve chief items which enter into water-works construction, covering the 26 years 1884 to 1909, inclusive. From these data it is possible to construct, by an operation familiar to statisticians, a series of numbers, one for each year, which will indicate the *relative* cost of a water-works as compared with any other year. The par-

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*Woodburn, Ind.

agraphs immediately following will be given in explanation.

Comparing the prices of any year with those of the previous year, some items would go up in price, some down, and others remain practically the same. But if cast-iron pipe should rise 10% in price and lumber fall 10%, it should not follow that the one offsets the other. The former is 15 times as important in water-works construction as the latter. Consequently, *weights* had to be obtained, which were found by searching through the appraisals of three typical water-works—made by the Wisconsin Railroad Commission. Cast-iron pipe was given a weight of 32 out of a total of 75. That is to say, that out of each $75 of outlay in constructing a water-works, $32 was expended for cast-iron pipe. Common labor came next with 14. Other items, as lead, steel, hydrants, valves, lumber, brick, skilled labor and land were given weights ranging from 5 to 2.

It is not necessary in this connection to give in detail the tedious process of compiling these water-works index numbers. In the left-hand column of Table II below are weighted index numbers for water-works construction costs from 1884 to 1909, computed on the basis of 100. They are merely *relative* numbers, indicating the trend of particular prices, in which each item has an influence proportionate to its importance as an element in the construction. For instance, in 1900 the number was 101.4, and in 1901 it was 94.3. That is to say, a plant, which would have cost $101,400 in 1900 could have been built the next year for $94,300 or for about 7% less.

Now do the relative numbers, or index numbers, for water-works construction costs rise and fall with those for cast-iron pipe prices? They do after a fashion, but it can hardly be said that they move together satisfactorily from year to year. If these two series of numbers had fluctuated together, then a 10% rise in the price of cast-iron pipe would mean a 10% rise in the appraisal value of the water-works. But such is not the case.

The same can be said for all commodities, as will be observed by comparing the numbers for all commodities with those for water-works. They do not always rise and fall at the same time, or in a close proportion.

TABLE I. RELATIVE PRICES OF CAST-IRON PIPE AND OF ALL COMMODITIES, 1884 TO 1909

Year	Cast-iron pipe	All commodities
1884	118.6	110.6
1885	99.7	100.6
1886	105.1	100.7
1887	118.0	101.9
1888	102.2	103.7
1889	99.5	106.2
1890	114.7	101.5
1891	94.0	100.4
1892	90.5	95.4
1893	85.5	94.9
1894	68.5	86.4
1895	83.2	84.1
1896	73.9	81.2
1897	62.6	80.6
1898	62.2	83.9
1899	101.3	91.4
1900	106.7	99.3
1901	91.8	97.5
1902	120.9	101.5
1903	121.1	102.1
1904	97.6	101.4
1905	108.8	104.2
1906	121.1	110.1
1907	141.7	116.4
1908	104.4	110.4
1909	103.6	113.7

TABLE II. INDEX NUMBERS FOR WATER-WORKS CONSTRUCTION COSTS AND COMPROMISE NUMBER FOR COSTS OF CASTIRON PIPE AND ALL COMMODITIES, 1884 TO 1909

Year	Water-works construction costs	Cast-iron pipe and all commodities
1884	109.1	114.6
1885	97.0	100.2
1886	102.1	102.9
1887	106.9	110.0
1888	99.2	103.0
1889	96.9	102.8
1890	103.5	108.1
1891	95.9	97.2
1892	91.9	93.0
1893	90.0	90.2
1894	77.7	77.4
1895	81.6	83.4
1896	76.6	77.5
1897	70.6	71.6
1898	74.0	73.0
1899	96.0	96.3
1900	101.4	103.0
1901	94.3	94.7
1902	112.2	111.2
1903	113.8	111.6
1904	105.4	99.5
1905	111.5	106.5
1906	122.2	115.6
1907	129.2	129.0
1908	110.8	107.4
1909	112.9	108.6

Since neither of the above move with the water-works numbers satisfactorily, a series of numbers half way between them was made. The right-hand column below is a compromise of the relative prices for cast-iron pipe and for all commodities. The relative number for cast-iron pipe in 1884 was 118.6, and for all commodities 110.6. Therefore, the compromise is 114.6.

It will be readily observed that the compromise numbers rise and fall in unison with those for the water-works costs, especially during the twelve-year period from 1891 to 1903. Even in the other years, for instance, from 1904 to 1909, they rise and fall consistently, or in proportion, which, after all, is the chief matter.

The deduction is, therefore, that the valuation of a water-works will change from year to year as the *combined* change in the price of cast-iron pipe and all commodities.

Practical Application

Let us now apply this truth to a concrete case, a water-works appraised at $300,000 in 1909. The index number for all commodities in that year was, say, 121.0, and the price of cast-iron pipe $27 per ton. The index number for all commodities in the present year is, say, 133.2, and pipe $25.50 per ton. All commodities have risen about 10% and cast-iron pipe has fallen about 6%. Therefore, the *combined* change is a rise of 2%. The plant whose valuation was $300,000 in 1909 would be $1.02 \times \$300{,}000 = \$306{,}000$ in the present year.

Of course, in the above computation, depreciation must be taken into account. The life of a water-works is usually considered fifty years, which is a depreciation of 2% per year. In the earlier part of its life it is less than 2% and in the latter part it is more.

No difficulty should be encountered in obtaining index numbers for all commodities. Probably the best one is compiled by the Bureau of Labor Statistics (Washington, D. C.) in its bulletins on Wholesale Prices. Another is compiled by Bradstreet. The one used above is of the author's own making. Any reliable index number is entirely satisfactory.

This method of arriving at the present valuation of a water-works may be used in case either an appraisal has been made within the last ten or twelve years or the plant itself was built within, say, fifteen years, as is the case in some of the young cities. Of course, it would not likely come as near the truth as a new valuation, but this would be, for most part, on account of the depreciation feature.

Here is an instance where the above method would certainly have merit. Two distinct appraisals are made of a water-works in the present year: one, $320,000, the other, $360,000. Eight years before an appraisal had been made also, which, by the use of the method offered here, is brought up to date, and found to be $355,000. It is strong evidence that $360,000 is much nearer the true value of the plant than $320,000.

Tests of Oil-Burning Locomotives are to be made by the Indian Government on the North-Western State Ry., according to the "British and South African Export Gazette" (London, England), for August, 1913. The trials will consume 18 months, the first six being devoted to the testing of different types of burners submitted and the last twelve to comparing the relative efficiency of oil and coal. Three locomotives for passenger trains and three for mail trains will be used in the tests. Contracts for about 6000 tons of oil have been placed.

The Summit Cutoff of the Lackawanna Railroad

SYNOPSIS—West of Scranton, Penn., the line of the Delaware, Lackawanna & Western R.R. is being thoroughly revised for a distance of 41 miles, by the construction of a new cutoff which includes some of the heaviest railroading ever undertaken in the East. This article gives details of the new line and its structures and describes some of the construction methods in use.

For some years the Delaware, Lackawanna & Western R.R. has been engaged in a systematic improvement of the alignment and profile of its main line between Hoboken, N. J., and Buffalo, N. Y. Built many years ago, primarily as a coal road carrying the product of its own mines, the road was originally intended only to connect, at a minimum expense, the coal districts of central Pennsylvania to the Great Lakes and Atlantic Ocean. The demands of modern economical railroading, however, have made desirable a number of radical changes in line and grade, which are being taken up in order by the company.

Up to the present the most extensive revision that has been undertaken is the well known Slateford-Hopatcong Cutoff, which was described in ENGINEERING NEWS, Aug. 13, 1908, p. 176. This line, just beyond the commuting zone outside New York City, was put through entirely new territory, substituting for a 39.5-mile line, with severe grades and curves and several tunnels, an easy grade, fairly straight line, 28.5 miles long and with only one tunnel. The old line was retained as a branch connecting some towns through which it passed. The work cost about $11,000,000. It contained some of the heaviest railroad construction ever attempted in line revision, notably a 110-ft. fill totaling over 6½ million yards of material. The structures on the line were all of concrete and included the very large concrete arch bridges across the Delaware River and Paulin's Kill.

The latest reconstruction which the Lackawanna has undertaken is the Summit Cutoff, which revises the line and grade of the main line just west of Scranton. This work is of greater cost than the Hopatcong Cutoff, being estimated at about $12,000,000, but is of somewhat different type. It is longer than the Hoptacong Cutoff, but it is entirely a replacement line, being for most of its length in sight of the old line for which it is substituted. The radical reduction of grades and curvature is achieved by very heavy cutting and filling and by viaducts of enormous size, all of which were inpossible in the early days of the railroad.

The Summit Cutoff is located on the Scranton Division, and consists of a pronounced revision from

FIG. 1. MAP OF PORTION OF PENNSYLVANIA AND NEW YORK, SHOWING PROPOSED AND ACCEPTED ALIGNMENTS OF SUMMIT CUTOFF, D., L. & W. R.R.

Clark's Summit, 7 miles west of Scranton, to Hallstead, 41 miles farther west. The accompanying map and profile, Fig. 2, show the old and the new line. The road between Scranton and Elmira is a portion of the main line division and freight trains operate between the two points as terminals. The heavy west-bound freight over the division originates at Scranton, coming there from the various coal fields in that district. Prior to 1910, all coal and dead freight was switched at Hallstead, turnaround runs being operated between Scranton coal districts and Hallstead, thus providing tonnage at the latter point when filling out west-bound trains and the extra power necessary for the pusher service over the heavy

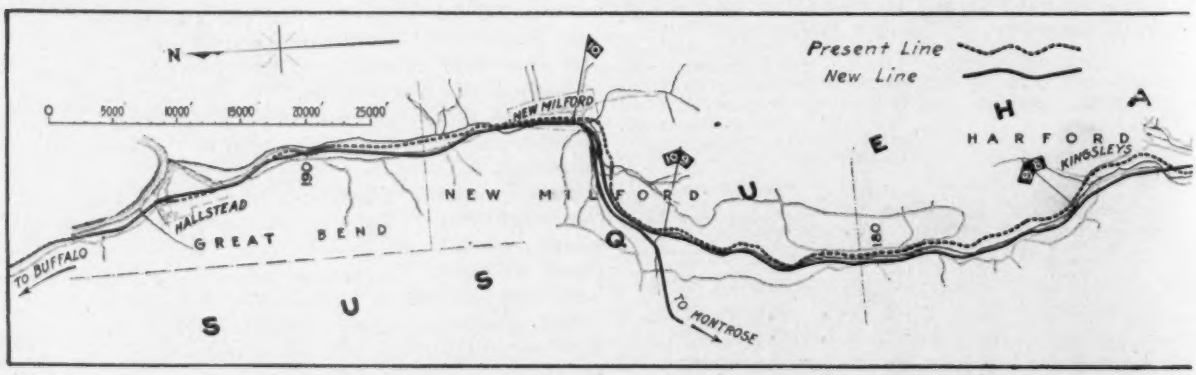


FIG. 2. PLAN AND PROFILE OF SUMMIT

grade east from Hallstead. This operation required a yard at Hallstead with the usual terminal facilities, and necessitated a considerable delay to all through trains.

In 1910 a hump yard was completed and put in operation at Scranton. This yard was built to size and shape up coal and dead freight trains for both east and west movement out of Scranton. Since the construction of this yard, trains have been run through the yard to Elmira, Syracuse and Utica, with no long delays other than that required to clean ashes and take coal at a new plant built for this purpose at Binghamton, thus eliminating the delay to road crews at Hallstead.

It became desirable after the construction of this yard and the instituting of a through run from Scranton to Elmira, to reduce the grade and curves on the Elmira-Scranton line, so that a minimum of pusher service would be required. Fig. 2 shows that the present line, after ascending from Scranton to Clark's Summit on a 75-ft. grade, descends to the south branch of Tunkhannock Creek at La Plume on a 65-ft. grade; then ascends on a 28-ft. grade to the Factoryville Tunnel. This tunnel is 2200 ft. long. It takes the line through the ridge between the south and the main branches of Tunkhannock Creek. West of the tunnel, the line again descends on a 65-ft. grade to Tunkhannock Creek which it crosses at low level at Nicholson. It then ascends along the east bank of Martin's Creek to New Milford Summit on a 21-ft. grade. From New Milford Summit to Hallstead, the line descends along Salt Creek to the Susquehanna River on a grade varying from 30 to 47 ft. to the mile. Thence west to Elmira the maximum grade in either direction is only 12 ft. to the mile.

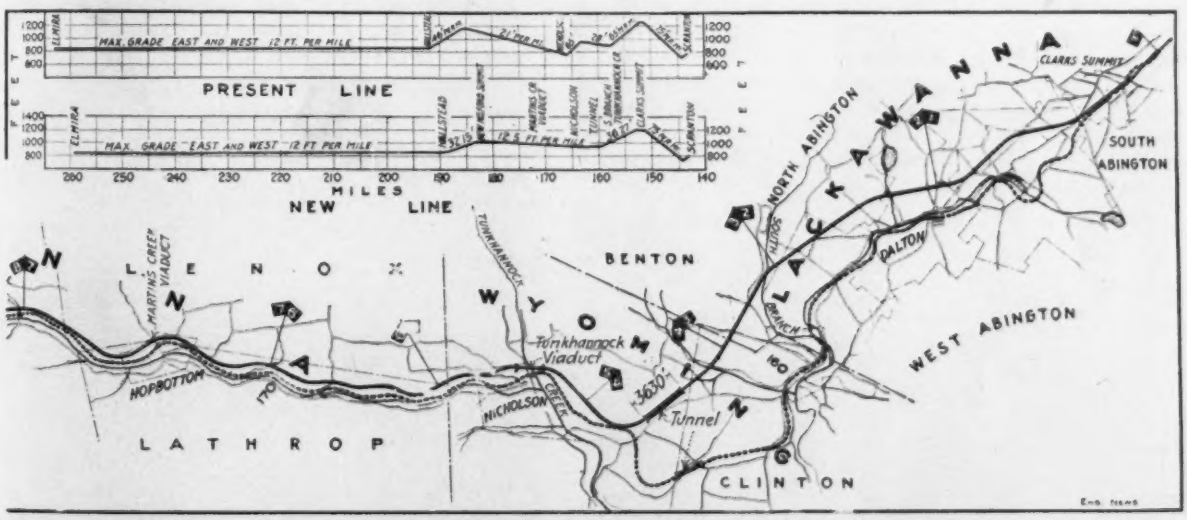
As a part of the original construction of the road this old line has sharp curves and heavy grades. Practically 50% of the line between Clark's Summit and Hallstead is curved, and many of the curves are between 4° and 6°. The total curvature in this distance of 43 miles is 3940°. Besides, none of the grades noted for the old line are compensated for curvature.

In relocating, it was desired to eliminate all westbound pusher grades west of Clark's Summit and also to reduce the east-bound grade from Hallstead east so that any class of engine in this service could handle with a pusher as heavy a train as it could bring over the line west of Hallstead. By a comparison of the tonnage handled on other parts of the road with similar operating and physical conditions, it was found that the east-bound grade should be reduced to 36 ft. per mile, if it were possible. The maximum allowable curvature was fixed at 2°.

With the above basis as a guide, the new line was finally decided upon after a series of surveys extending over a period of three years. The country through which the cutoff runs is extremely rough, made up of a series of hills and valleys irregularly broken up. Any line following a fair grade would, with a minimum of cutting, have been impossible as to curvature, and fair alignment demanded the heaviest earthwork and in some cases, very long and deep tunnels. No government contour maps were available for the territory and the work of location was extremely tedious.

Fig. 1 is a map of the section of country through which the line passes, and on it are indicated the four preliminary locations decided upon after over 300 miles of line had been run. One of these lines, the one to the extreme west, was a complete relocation. It shortened the main line from Scranton to Elmira by 21 miles and could have been run with very desirable grades and curvature, but it required the heaviest kind of construction and furthermore passed through practically virgin country far away from the main centers, Binghamton and Owego, with the Syracuse, Utica and Ithaca connections. Mainly on the latter account it was discarded.

The line shown to the extreme east gave by far the most satisfactory grades and alignment and retained the necessary Binghamton entry, but it was not accepted on account of a 7000-ft. tunnel which was so far below surface as to require it to be driven from both ends through unsatisfactory rock (and bad drainage conditions). This left two alternatives, as shown, one immediately to the east and one to the west of the old line. For reasons of superior line and grade, the former was chosen. Work on this line was started about a year ago. It is expected that it will be completed in about two years.



CUTOFF, SHOWING OLD AND NEW LINES

The new line is shown in detail on Fig. 2. It leaves the old line just east of Clark's Summit station, parallels closely the old line for about one mile, and then cuts across country on a succession of heavy cuts and fills for about 10 miles in entirely new location and, as noted on the profile, at a much lower grade. In this section is most of the heavy work of the whole cutoff, for the new location is boldly driven through the many hills and valleys which the old line swung around, as the many curves shown in Fig. 2 would indicate. Practically all of the 3½ miles of length saved is made in this first 10 miles. In this section, too, is the only tunnel on either old or new line, 2200 ft. long on the present line and 3600 ft. long on the revision.

At about Sta. 500, that is nearly 10 miles from Clark's Summit, the line touches the old alignment at a higher elevation and from there on to Hallstead, where it coincides with the old line in both line and grade, the new line is parallel to the old and always within a few hundred feet. After cutting through the ridge bordering Tunkhannock Creek, about a mile west of the west portal of the tunnel, the new line crosses the creek on the enormous Tunkhannock Creek Viaduct, which was described in ENGINEERING NEWS, June 13, 1912, p. 1110, at an elevation of 240 ft. above the bed of the stream or 183 ft. higher than the adjacent old bridge.

The line then passes into the Martins Creek valley, along which it runs to the east of the old line and in side-hill cut and fill along the side of the slopes above the old line. Martins Creek and the old line are crossed by the Martins Creek Viaduct, 150 ft. above the creek bed and 88 ft. above the old line grade. From there north to Hallstead the line parallels the old. The earthwork in this section, mostly in side-hill, is small compared to that on the first ten miles of the cutoff and no large structures are required.

The old line had 22 highway crossings at grade, all of which were marked for elimination in the near future. These are all taken care of in the new construction, which has no highway crossing at grade. In many cases this required substantial revision of adjoining highways, at expense which devolved upon the railway company.

Table I gives a brief statement of the difference in grade and alignment between the new and the old line.

TABLE I. SUMMIT CUTOFF, D. L. & W. R.R. OLD LINE-NEW LINE SAVING

	Old	New	Saving
Length of line	43.2 mi.	39.6 mi.	3.6 mi.
Maximum grade (eastbound)	1.23% (uncompensated)	0.68%	0.55%
Maximum grade (westbound)	0.52% (uncompensated)	0.237%	0.283%
Rise and fall	553.00	226.00	327.00
Maximum degree curvature	6° 22′	*3° 00′	3° 22′
Total degree curvature	3940	1500	2440

* Very few curves exceed 2° on the new line.

It will be noted from Fig. 2 that the new line retains the 75-ft. ruling grade westbound out of Scranton. Scranton is in a deep valley with rising grades on either side which cannot be avoided without excessive curvature and length of line. A pusher grade and service is therefore necessary up from Scranton to Clark's Summit.

The existing line from Scranton to Elmira is for most of its length a two-track line. One of the governing reasons for the construction of the cutoff was the fact that in the near future it would soon have become necessary to build an additional third track along the present alignment, especially on the heavy grades. The new line will be three-track between Clark's Summit and New Milford, except through the one tunnel and across the long Tunkhannock Creek Viaduct, where only two tracks are provided for. From New Milford to Hallstead, the present two tracks, with some slight revision in alignment, will remain for west-bound movement, and the new line will consist of two tracks for east-bound movement.

The Chief Engineer's office has given out the following complete list of items considered in calculating the amount of money which could be spent in reconstructing the line, and have the decreased cost of operation and maintenance, due to the reduction of distance, elimination of curvature, rise and fall reduction of gradient meet the interest charges.

1. The total cost of operation and maintenance for the year 1910.
2. Total train miles for 1910, exclusive of work train mileage.
3. Cost of operation and maintenance per train mile; that is, Item 1, divided by Item 2 = $1.67.
4. Average yearly increased cost of operation and maintenance per train mile from 1900 to 1910 = 4.4%.
5. Average yearly increase in net tons of freight handled, 1900 to 1910 = 7.425%.
6. Average yearly increase in passengers carried, 1900 to 1910 = 8.2%.
7. Total net tons of manifest freight, moved over the territory in question, 1910. East, 1,324,907. West, 592,713.
8. Total net tons of all classes of freight moved over the territory in question, 1910: East, 3,134,688; West, 6,967,269.
9. Total number of freight cars moved over the territory in question, 1910: East, 188,237 loads; West, 143,279 loads; East, 122,919 empties; West, 123,146 empties; total tons East, 8,306,003; total tons West, 12,066,973.
10. Passenger and milk trains, east and bestbound = 26.
11. Total reduction of distance.
12. Elimination of curvature (total degrees).
13. Elimination of rise and fall.
14. Reduction of gradient.

CONSTRUCTION

For purposes of construction the cutoff has been divided into 10 divisions, as shown in Fig. 2, and with minor exceptions contracts were awarded about a year ago coterminous with these divisions. Table II shows the contractors for the various parts of the work.

TABLE II. CONTRACTORS ON SUMMIT CUTOFF, D. L. & W. R.R

Contract	Name	Address
1	Robert Grace Cont. Co.	170 B'dway, N. Y. C.
2-3	Reiter, Curtis & Hill	Arcade Bldg., Phila., Pa.
4	David W. Flickwir	Roanoke, Va.
Part 5 incl. Tunkhannock Viaduct*	Flickwir and Bush, Inc.	No. 1 Mad. Ave., N. Y. C
Part 5 and Part 6	Walts & Reece Consn. Co.	Billings, Mont.
Part 6	James A. Hart	256 B'dway, N. Y. C.
7	T. Burke	Connell Bldg., Scranton, Pa.
8 incl. Martins Creek Viaduct	F. M. Talbot Co.	No. 1 Mad. Ave., N. Y. C
9	P. McManus	Arcade Bldg., Phila., Pa.
10	W. H. Gahagan	189 Montague St., Bklyn N. Y.

*Grading on east end of section 5, sublet to John Burke, Scranton, Penn. Grading on section 8, sublet to John Burke, Scranton, Penn.

Each contractor has taken the full amount of work on his section, including all cleaning and grubbing, excavation, fill, new road construction, masonry, etc. As a rule bids were accepted on unit quantities, excavation being separated only into two classes, earth and rock with no overhaul, and concrete into three classes: 1:2:4 for reinforced floors and concrete girders, 1:3:5 for abutments, arch rings, etc., and 1:3:5 with large stone for cyclopean masonry in foundations and massive piers.

The Cutoff is a succession of very heavy cuts and fills, with a number of minor masonry structures and two very large concrete viaducts, one by far the largest that has ever been attempted in concrete. It was not possible to balance the cuts and fills without spoiling the align-

ment, so that the cuts somewhat overbalance the fills, the extra spoil being taken to near-by spoil piles.

The magnitude of the cut-and-fill operations will be shown by the following data, taken from a preliminary estimate and not, therefore, precisely accurate. The total earth excavations for line cut will be about 5,000,000 cu. yd. and the total rock excavation for line cut about 7,600,000 cu.yd. In addition to this, there will be about 400,000 cu.yd. of earth excavation in connection with realignment of highways and masonry foundation excavation, and about 90,000 cu.yd. of rock excavation for the same purposes. This gives a grand total of about 5½ million yards of earth and 7½ million yards of rock excavation along the 40 miles of realignment. The concrete for the structures will total about 300,000 cu.yd.

An idea may be gained of the magnitude of these operations also by the tabulation (in Table III) of the estimates for the large cuts and fills on the work. It will be seen that the maximum fill is 1,600,000 cu.yd. over a length of 2000 ft. and a maximum depth of 142 ft. The largest cut is about 1,450,000 cu.yd., and a maximum depth of about 116 ft.

CUTS—Practically all of the cuts along the line are being taken out with steam shovels emptying into narrow-gage cars drawn by locomotives back into near-by fills or spoil banks. Where rock is found, the large-bore well drill is the most generally used method for making blast holes, although some contractors are using the small-bore air or steam drill.

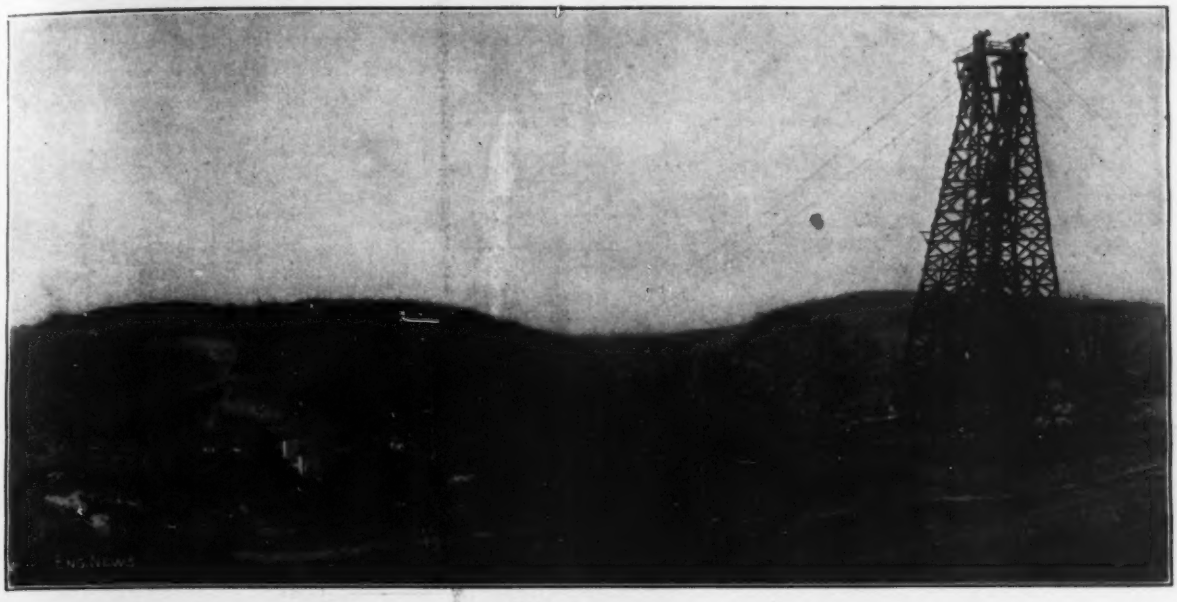


FIG. 3. VIEW OF TUNKHANNOCK CREEK VALLEY, SHOWING VIADUCT UNDER CONSTRUCTION

FILLS—Standard embankments on the line are on a 1½ to 1 slope with a 46-ft. top width for a three-track line.

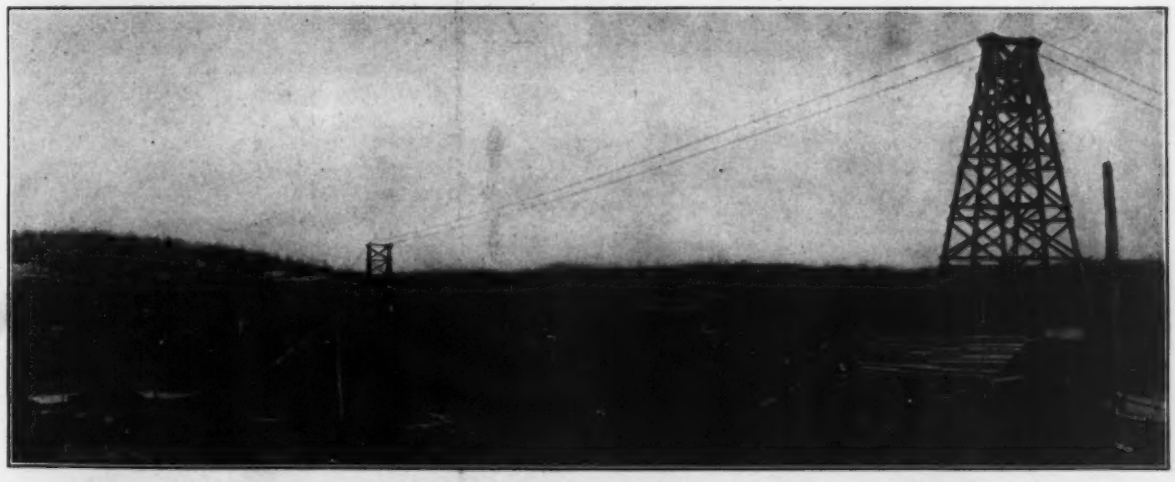


FIG. 4. SUSPENSION BRIDGE DUMP MAKING ACKERLY FILL, SUMMIT CUTOFF, D., L. & W. R.R.

Three general methods of making embankments are being employed: the ordinary trestle, the suspension-bridge dump and a special balanced-train system in use on the 142-ft. South Branch fill.

The trestle method, of course, needs no description here. It is sufficient to say that in by far the greater number of fills the conditions are such that this simple method is the best and cheapest that could be devised. In three of the deepest fills, the Ackerley, Lake Sheridan and station 206-25 (see Table III), the suspension-bridge dump so successfully used on the Hopatcong Cutoff is

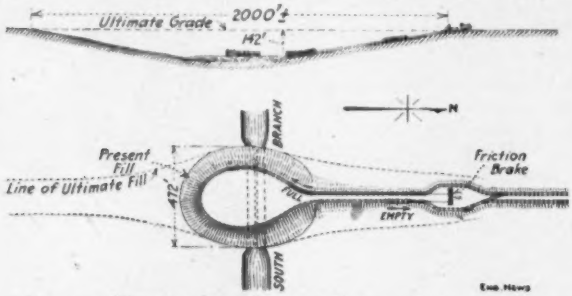


FIG. 5. METHOD OF MAKING SOUTH BRANCH FILL

employed. This device was described in ENGINEERING NEWS, Apr. 22, 1909, p. 423.

As described there, the device consists of two towers, one fixed and one movable, between which a double cableway is suspended. From this cableway is hung a cradle, or traveling bridge, on which a 3-ft. gage track is laid. Beginning just inside of the movable tower a construction track is started on the line of fill, onto which the dump cars are pushed. As the fill progresses, this track is extended out over the tailing, being suspended from the cradle, which is moved ahead with the fill. As the fill is made, the cableway span is shortened by moving the movable tower, or by providing an intermediate tower which can be moved along as required. The longest suspension dump had an original span of about 1900 ft. A view of the one at Ackerley Fill is shown in Fig. 4.

The extreme width and depth of the South Branch fill (Sta. 290-310) has made possible the use of a somewhat novel method of filling, which is roughly outlined in the sketch in Fig. 5. The fill has a maximum depth of

TABLE III. LIST OF LARGEST CUTS AND FILLS, SUMMIT CUTOFF, D., L. & W. R.R.

Name	Cuts Sta.	Cu.Yd.	Max. Depth
Clarks Summit	—50–50	1,450,000	
Watkins Hill	80–110	220,000	
Casey	114–140	390,000	
Slayton	245–265	420,000	1[illegible]
Sisson	273–290	320,000	
Crisman Cut	321–373	940,000	[illegible]
East Tunnel	404–423	680,000	
West Tunnel	460–477	360,000	
Rought Cut	492–503	260,000	
East Viaduct		505,000	
Pratt Cut	653–677	290,000	
Shick Cut	706–725	600,000	
Capwell Cut	739–778	350,000	
Bell Cut	814–827	250,000	
Roberts Cut	872–880	230,000	
Simons	935–962	210,000	
West of Martins Creek Viaduct	1062–1114	480,000	114
Decker Cut	1330–1360	300,000	
New Milford Summit Cut	1530–1556	300,000	
Blakeslee Cut	1678–1685	573,000	

Name	Fills Sta.	Cu.Yd.	Max. Hgt.
Ackerley	50–80	830,000	115
Dalton	140–160	570,000	110
Smith	203–221	540,000	120
South Branch	290–310	1,600,000	142
Winters Fill	390–404	660,000	
Quarry Lot Fill	477–490	243,000	
Jayne Fill	504–517	290,000	
East of Tunkhannock Viaduct		572,000	
Zeck Fill	640–655	203,000	
McKeon Fill	677–706	500,000	110
Riker Fill	722–738	500,000	
Bell Fill	799–813	300,000	
Woodruff Fill	888–905	400,000	
Martins Creek Viaduct Fill	1050–1063	450,000	
Decker Fill	1377–1397	220,000	
McConnell Fill	1417–1460	400,000	
Cobb Fill	1480–1540	64,000*	

* Already 300,000 cu.yd. placed and not filled.

142 ft. over the South Branch of Tunkhannock Creek, which is carried through the embankment by the two-barreled arch culvert described elsewhere in the article. This depth gives a maximum bottom width of 472 ft., the whole yardage across the 2000-ft. length being 1,600,000 cu.yd. The fill is being made from the next northerly cut, a mixture of rock and earth with rock predominating, and the spoil is brought to the northerly limit of the fill in dump cars running behind a locomotive on a narrow-gage track.

At the top of the slope to be filled there has been erected two winding drums on the same continuous axle, with cables wound in opposite directions. The narrow-gage tracks from the cut pass around the winding drums

FIG. 6. MAKING THE 146-FT. SOUTH BRANCH FILL

(See text for description of method. Circular tracks from which fill is made in middle distance, gravity tracks in background leading to wire-rope spools just back of trestle, which is being used to make upper part of fill by ordinary dump methods. Culvert carrying creek immediately in front of camera, but not in view.)

and descend the slope in line with the winding drums. In filling, a train-load of earth or rock is pushed in by locomotives around the drums to the down-hill track. When the cars reach the tangent in front of the drum, a cable from that drum is attached to the train, and the locomotive detached and run back on the level to pick up the empties from the fill and to take them back to the cut. Meanwhile, a train of empties down on the fill is attached to the end of the cable from the other drum. The loaded train is then allowed to descend the slope by gravity, pulling the empties up the other track, the speed and general progress of the cars being controlled by a friction brake on the common axle. When the full train reaches the bottom it is picked up by a locomotive, drawn around the fill on the tracks, laid close to the slope (as shown in Fig. 6) dumped at the proper place and then backed to the foot of the slope and drawn up as an empty by the descent of a full train on the other track.

The inclined tracks are laid on the natural slope and the fill tracks are moved as occasion demands to fit the requirements of the fill. The layout has proved quite successful and economical. It is on the Reiter, Curtis & Hill contract.

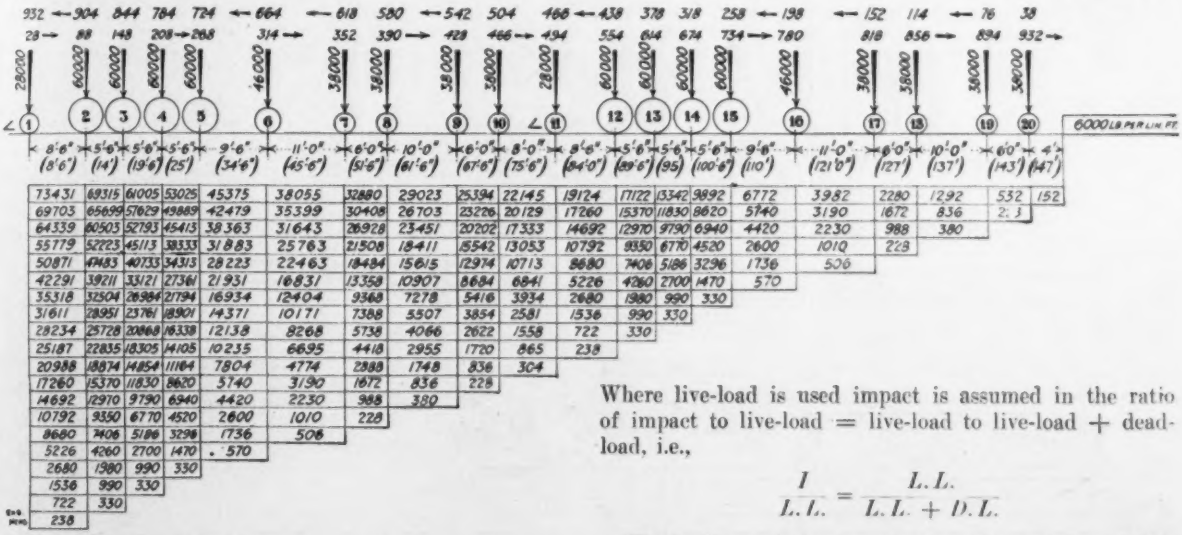


FIG. 7. D., L. & W. R.R. STANDARD SPECIFICATIONS, MIKADO TYPE LOADING-2-E-60

MINOR MASONRY STRUCTURES

There are 85 minor structures on the entire line and out of this number 78 will be built of concrete. This includes all culverts, highway arches and viaducts. A number of different types of structures have been designed for the varying conditions. These may be outlined as follows: For small drainage openings, arch culverts with reinforced inverts are used varying in size from 3 to 10 ft. For larger streams and highways, the arches vary from 20 to 34 ft.; in some cases the stream flow is so great as to require double-barreled arches. For spans from 20 to 24 ft. where clearances forbid the use of arches, flat topped under-crossings are built. In general these are of flat-slab type, but in some cases a T-beam bridge has been designed.

Some overhead crossings are required in cuts and at these locations generally overhead arches are used 60 ft. in span and with vertical clearance over the outside track of 21 ft.

DESIGN—In designing all arches under embankments the entire fill over the crown is assumed as acting vertically and no allowance is made for the arch action of the fill. In determining foundation pressures the horizontal thrust of the entire depth of fill against the back of the abutment is combined with the vertical load over the abutment and with the arch thrust. In high fills, such as are shown in Fig. 8, the live-load of the train is not considered in designing the arch, but the varying weight of the load due to the side slope is assumed to reach the arch ring vertically. In such cases the thickness of the arch ring is changed, as shown in Fig. 8, to meet the requirements of the lessening dead load. At the same time the abutments are narrowed for the same loading. In the lower fills the live-load of the train is considered to pass down through the fill on a slope of ½ to 1 at right angles to the track and the abutments and arch rings are reduced at the point of intersection of this assumed load line to meet the lessened requirements.

Where live-load is used impact is assumed in the ratio of impact to live-load = live-load to live-load + dead-load, i.e.,

$$\frac{I}{L.L.} = \frac{L.L.}{L.L. + D.L.}$$

The loadings on the structures on the Cutoff are taken from the new Mikado type D. L. & W. engines and the Mallet compound type. These give about the same moment diagram, but the Mikado type is generally somewhat in excess. The loading diagram for this type, known as "Mikado Type Loading 2E60" is shown in Fig. 7. It will be noted that this is an exceedingly heavy load.

Figs. 8 to 12 show some of the typical minor structures along the line.

DOUBLE-BARREL CULVERT—Fig. 8 shows bridge No. 1.17, a typical double-barrel culvert. In locations where the culverts had to be built in deep fills on an earth foundation a reinforced invert was used as shown. The sections of the ring in the invert were decreased toward the end in the manner described above, but the foundation pressure was determined by assuming a load for the entire fill acting vertically over the culvert and proportioning the bottom slab for the entire upward pressure. Single-barrel culverts are designed in a similar manner. In this reinforced design stresses of 16,000 lb. per sq. in. tension in steel 650 lb. per sq.in. compression in concrete per sq.in. were used.

SINGLE HIGHWAY ARCH—Fig. 9 shows highway arch bridge No. 2.38. This is typical of single arch construction under a light fill, where the live-load is the determining factor. The arch is 34 ft. semicircular on right section, but the loading crosses it on a 21° 57′ skew. The live-load is assumed as spreading through the fill on a slope of ½ to 1 from the edge of the ties to the crown of the arch. This determines the length of the center section and the remainder of the arch is designed for maximum height of fill over it with no live-load. The section of ring in this particular case was made uniform throughout and the abutment only changed for end section. If the arch had been longer, both ring and abutments would have been reduced toward the end, as noted in the paragraph above.

For the center section two loadings were analyzed, a live-load over the entire span and a live-load over half span, with impact as noted above. The arch was designed as a plain concrete section, but the reinforcement steel, ¾-in. square bars 2 ft. c. to c., was placed in the ring as an additional factor of safety. There was no tension in the ring for live loading. It will be noted that the skewbacks for this arch are laid on the skew so that the section of each 2 ft.-ring is a right arch of 36 ft. 8 in. inside span, whereas the true right-angle span of the arch is only 34 ft. The bridge was designed as a 36-ft. 8-in. right span, and was built in the separate skewback sections as shown.

FLAT-TOP SLAB UNDER-CROSSING—Fig. 10 shows a typical flat-top slab under-crossing, bridge No. 20.51. This is typical of the flat-top under-crossing where clearance over roadways is so limited as to make the arch type unsuitable. The regular live-load design was assumed here spreading over 13-ft. widths of slab, which is the distance between centers of track. The top of the rail is 2 ft. above the top of the slab, which is the minimum depth used for track and ballast.

T-BEAM FLAT-TOP SLABS—Fig. 11 shows bridge No. 1.01, a typical T-beam under-crossing. In locations where a slightly greater road clearance was available than for bridge No. 20.15, shown in Fig. 10, but where this clearance was still insufficient for arch construction the T-beam section for flooring was used. This reduced the dead-load and proved more economical than the slab of rectangular cross-sections. The design is very simple, as shown in the figure.

SINGLE-ARCH ROADWAY—Fig. 12 shows a typical single-arch roadway design. This does not differ from the design shown in Fig. 8, except that it was designed for its middle portion as a right arch and was not reinforced there. The skewed ends were built in sections of varying skews. These skew ends were also reinforced as shown on the drawing, whereas the center section was of plain concrete.

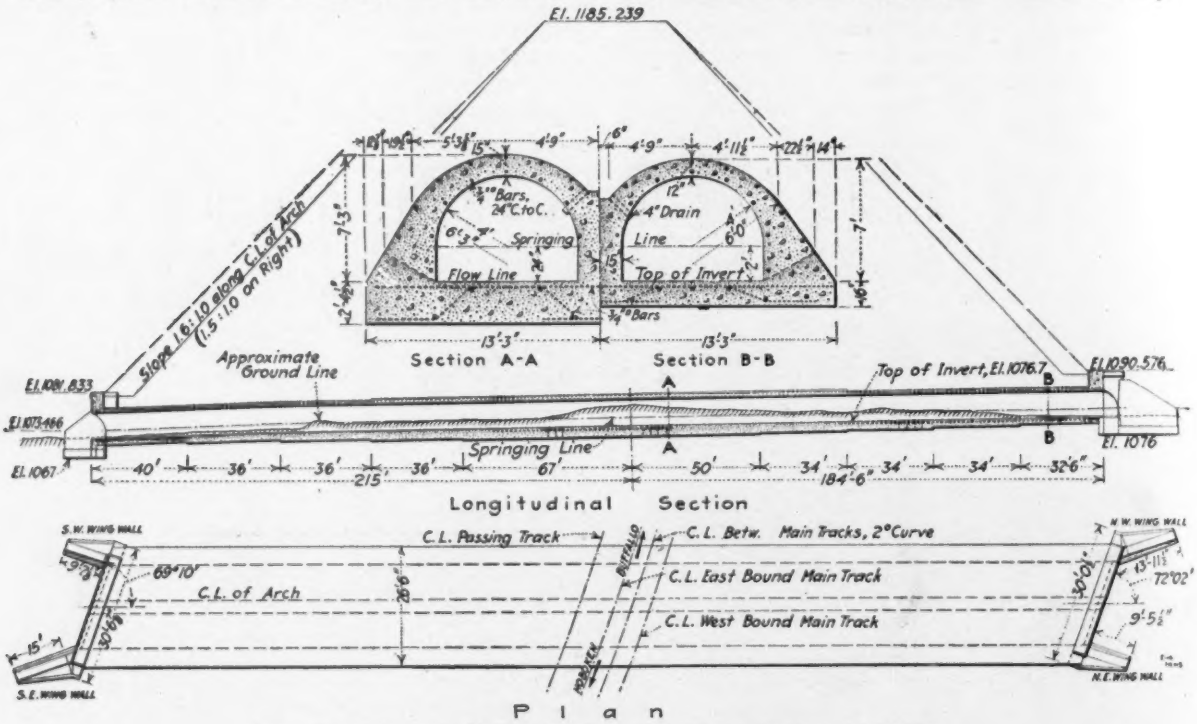


FIG. 8. TYPICAL DOUBLE-BARREL CULVERT. BRIDGE NO. 1.17

SOUTH BRANCH DOUBLE-BARREL CULVERT—One of the notable structures on the line, notable for its length, though not for its design, which is not very different from some of the others, is the long two-barrel culvert under the South Branch fill, which is described above. This culvert has to carry the heaviest fill along the line, 146 ft. The creek will be carried through the embankment at an angle of 68° to the center line of track. Two arch culverts, of 24-ft. span each, are provided for the 471-ft. barrel length.

Soundings along the stream showed rock at a depth of 53 ft. below the bed of the creek with a slope rising rapidly to the side-hill. To design the double-barrel arch for a heavy fill with the assumptions of loading which have been noted it would have been necessary to carry the foundations to rock. After a study of various

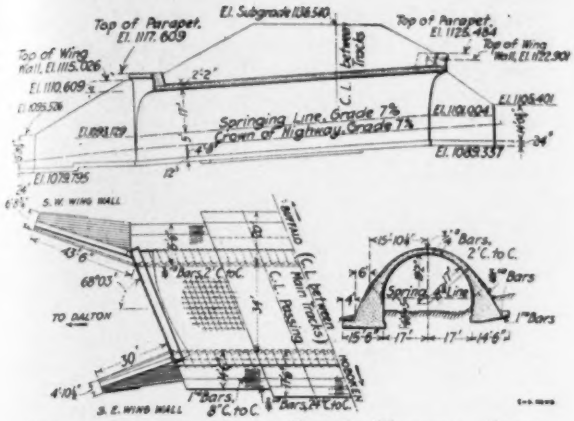


FIG. 9 TYPICAL SINGLE-BARREL HIGHWAY ARCH, BRIDGE NO. 2.38

schemes it was found to be more economical to change the channel of the stream to a location on the side-hill where the rock on the center line of culvert was at the same elevation as the present bed of stream. This requirde a heavy steam-shovel cut, but the concrete volume was reduced to a minimum.

The ring and abutments were reduced in four sections from the center to each end. The center section was designed to a maximum of fill, the length of which was determined by the regular slope of ½ to 1. The remaining length of the barrel was divided into three equal lengths of 54 ft. and each section designed for the greatest depth of fill over it. The crown thickness at the center section is 3 ft. 2 in., which decreases to 2 ft. for the end sections. The ring was proportioned so that the

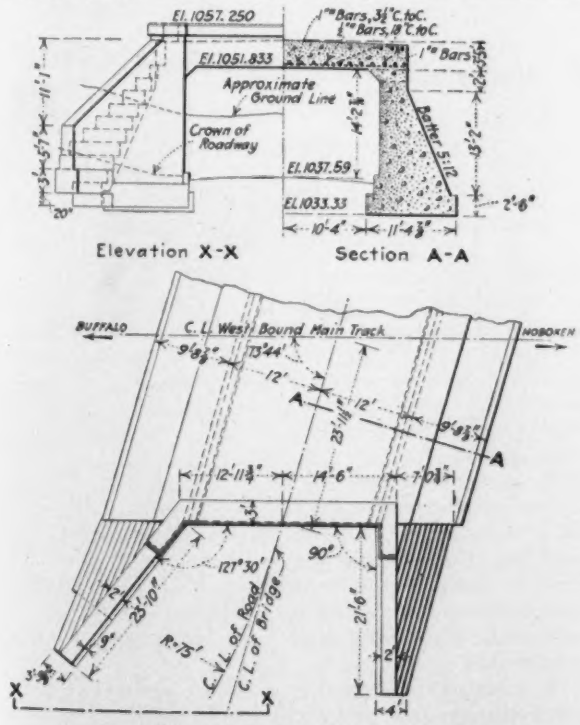


FIG. 10. TYPICAL FLAT-TOP UNDERCROSSING, BRIDGE NO. 20.51

line of pressure everywhere is within the middle third, producing no tension but a maximum compressive stress of 460 lb. per sq.in. No reinforcement is placed in the ring except at the end section which will be built on the skew.

In determining the length of this arch and of the other arches on the line the slope of the fill is assumed at 1½ to 1 and an additional length is given of 1½ in.

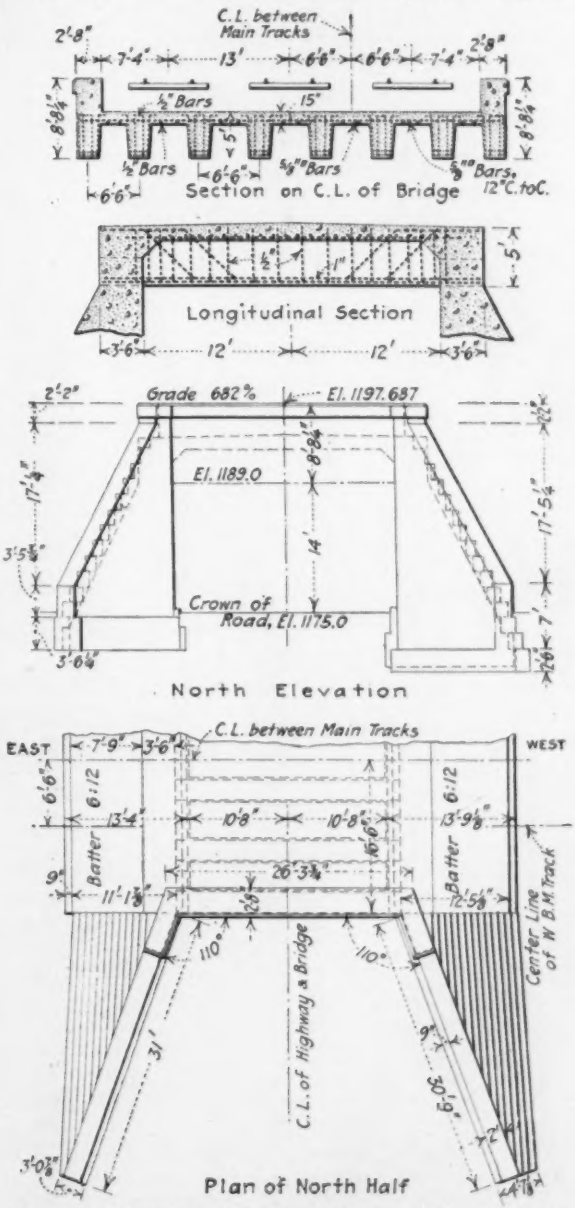


FIG. 11. TYPICAL T-BEAM UNDERCROSSING, BRIDGE NO. 1.01

per foot of fill over the crown to provide for overrun of fill.

LARGE VIADUCTS

In addition to the minor masonry structures noted above two very large viaducts are being built on the line. The Tunkhannock Creek Viaduct, the larger of the two, is being built at Nicholson, 32 miles west of Scranton,

Penn. Here the new line is almost parallel and only 410 ft. north of the present line. The viaduct is 34 ft. wide and will carry two tracks. It consists of ten main arches of 180-ft. span and two 100-ft. semicircular arches, which are covered by the fill. The main arches will carry a concrete superstructure composed of eleven 13-ft. 6-in. transverse spandrel arches. The length of this viaduct over all is 2375 ft. and it has a maximum height of 240 ft. over the bed of the creek and 300 ft. from the deepest foundation. Two piers are carried to rock in 95 ft. of excavation. The depth of rock decreases toward the end of the viaduct and at the abutment is only 10 ft. below the ground surface.

This viaduct will contain approximately 163,800 cu. yd. of concrete and 2,275,000 lb. of reinforcing steel. The earth and rock excavation will amount to 40,000 and 3500 cu.yd. respectively.

The Martins Creek Viaduct crosses the present main line at Kingley's, 32 miles west of Scranton, Penn. The main spans consist of two 100-ft. semicircular and seven 150-ft. three-centered arches with 59 ft. rise, which will support the track on a concrete superstructure composed of 12-ft. 6-in. transverse spandrel arches. There are two 50-ft. semicircular approach arches covered by the earth back fill. This viaduct is 47 ft. wide and provides for three tracks. The maximum height from the bed of creek to the top of rail is 150 ft. and 175 ft. from the deepest rock foundation to the top of rail. Its total length is 1600 ft. and it will contain approximately 77,500 cu.yd. of concrete and 1,600,000 lb. of reinforcing steel. Besides this, 21,900 cu.yd. of earth, and 4000 cu. yd. of rock will be taken out in the foundation excavation.

The accompanying views in Figs. 3 and 13 show the stage of the construction of the Tunkhannock Creek viaduct in May of this year. They give a very good idea of the character of the country through which the line passes and of the nature of the valley over which the viaduct runs. They do not, however, convey a proper idea of the immense scale of the structure. When it is considered that the tower shown in the central part of Fig. 13 is 260 ft. high it will be realized how enormous the Tunkhannock Creek Viaduct is.

Tunnel

The one tunnel along the line is located between Sta. 423 to 459 and is 3600 ft. long. This tunnel is driven through a close-grain sandstone, known as bluestone, which is the stone that is generally used for pavement flagging in the eastern part of this country. It is 30 ft. wide in the clear and will have a clear height of 23 ft. 5 in. from the base of the rail to the crown of the arch. It is of the regular horseshoe railway tunnel section. The central section will probably be driven through solid rock with no lining. Near the entrances where soft rock may be encountered, there will be placed bench walls of concrete 2 ft. thick with an arch of four courses of vitrified brick.

The tunnel is being driven with a center top heading both ways from two shafts located 900 ft. from each end. These shafts are 50x30 ft. in section and have reached a depth of about 130 ft., the elevation of the top heading.

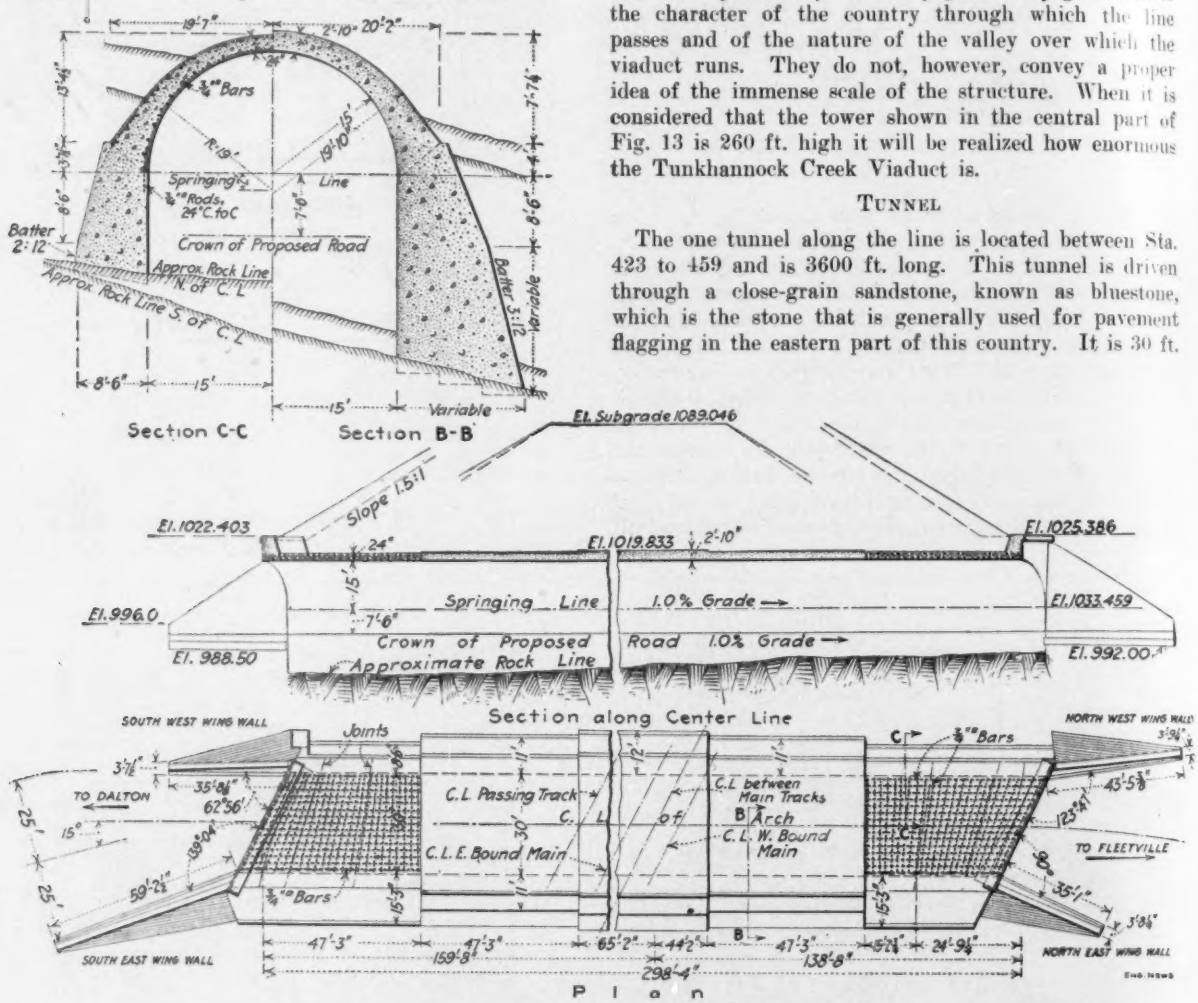


Fig. 12. Typical High-Fill Arch, Bridge No. 4.03

Personnel

The Summit Cutoff from its initial projection has been under the direction of George J. Ray, Chief Engineer, Delaware, Lackawanna & Western R.R. The construction is under the immediate supervision of F. L.

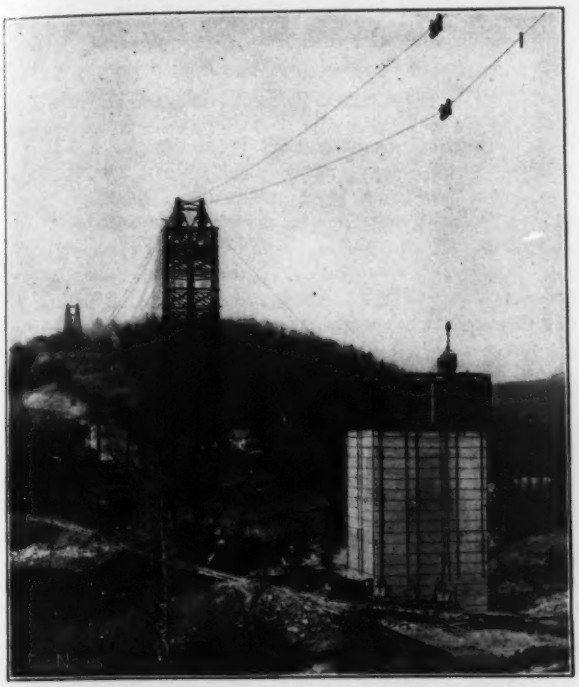


Fig. 13. View of Tunkhannock Creek Viaduct under Construction

Wheaton, Engineer of Construction, who had a similar relation to the Hopatcong-Slateford Cutoff. The design of the masonry structures is in charge of A. B. Cohen, Assistant Engineer.

Raising a Wrecked Atlantic Liner by Compressed Air

The use of compressed air in raising and repairing sunken vessels has been employed very extensively, either in floating caissons (or "camels") attached to the ship or by converting part of the ship itself into a pneumatic caisson. An exceptionally important piece of work done in the latter manner was the raising of the Atlantic liner "Royal George" in the St. Lawrence River. A specially interesting feature of this work was that the hull when raised was repaired under air pressure while floating in the river (instead of going into dry-dock). By this means the vessel was enabled to take cargo and cross the Atlantic to have permanent repairs made at its home port.

The "Royal George" is one of the steamers of the Canadian Northern Line, running between Montreal, Canada, and Bristol, England. It is 545 ft. long and 60 ft. beam, with a registered tonnage of 12,000 tons; it is driven by triple screws and steam turbines, developing a total of 18,000 hp. and giving a speed of about 20 knots. On Nov. 6, 1912, while on its last voyage of the season from Montreal (before the closing of the river by ice) it went ashore at high tide about nine miles below Quebec. Passengers and cargo were discharged and attempts made to pump out the flooded compartments of the hull. The following description of the salvage work as finally accomplished is condensed from an article by R. G. Skerrett in the *Scientific American*.

After nearly two weeks it was recognized that the usual salvage procedure would not answer, and W. W. Wotherspoon was summoned from his work on a nearby wreck (the stranded collier "Gladstone"). Upon this he was using compressed air, and the underwriters decided to have him try the same method upon the "Royal George."

RAISING THE SHIP

The pumps were removed and the hatches to the damaged compartments sealed by air-tight plates. Air-locks were secured to circular passageways in these hatch covers, and connections also made by pipe between the air compressors and the injured holds. In three days this work was completed.

It was necessary to provide against the pressure of a head of water of 23 ft., and the deck overlying the compartment was not equal to a bursting stress of this magnitude. Worse still, the deck planking did not rest upon a solid steel deck, and there was risk of blowing out the calking between the planks, which would mean failure. To meet the situation, hot pitch was poured into all the deck seams, the deck covered with a number of layers of tar paper, and over these placed a course of spruce flooring. Then this pressure deck was braced by shores reaching to the deck above, so as to distribute the stress when the holds should be filled with compressed air. In brief, it turned the flooded cargo spaces into caissons, and then all was in readiness for the expulsion of the water by forcing in the air under pressure. The compressors started at 10 p.m. of Nov. 22, and 15 min. later the forward cargo space was dry.

An internal examination of the injured holds showed that nearly 40% of the ship's bottom was damaged, but it also demonstrated that the salvage method was equal to the demands to be placed upon it. The next afternoon, two hours before high tide, the water was blown out from the flooded compartments and the vessel floated clear.

REPAIRING THE HULL UNDER AIR PRESSURE

Then came the question of repairing the badly damaged hull. The nearest dry-dock was at Montreal, 140 miles away, and the river's closed season was drawing near. Besides the liability of having the ship held for months, there was the prospect of heavy dock charges, independently of the cost of repairs. The question was "How could the 'Royal George' be made ready for sea without going into dry-dock?" Mr. Wotherspoon answered this in a unique and satisfactory manner.

The ship was anchored in deep water, and into the damaged spaces men were sent through the air-locks, the compartments being held substantially drained by reason of the compressed air within them. Beginning at the uppermost point of the rents in the hull, the men roughly sealed these openings by covering them with heavy planking calked with mud and oakum, the water receding as the "pudge-boards" reached downward and finally covered the openings. This sealing was but temporary and effective only so long as the air pressure was maintained. Before the vessel could proceed seaward it would be necessary to cover the damaged bottom from without with steel plates made water-tight.

From within the hull, flexible templates of wood were made of each needed patch, and upon these were marked the places for bolt holes, the same positions being indicated upon the inner surface of the vessel's plating. The templates were sent up through the air-locks and guided the steel workers in forming the steel patch plates and in boring the threaded bolt holes. Into these holes tap bolts were screwed. In the meantime, slightly larger holes were drilled around each wound in the hull, and made tight with wooden plugs from within.

A weighted platform was swung under the ship directly beneath the rents to be mended, and a diver took his place upon this submerged platform. When each patch was ready, with its tap bolts in place, it was lowered to the diver, who directed it into position, the men inside withdrawing the wooden plugs so that the bolts could enter their proper holes. Then nuts and washers, with plenty of red lead, were placed upon the bolts and screwed well home in order to make the patch-plate snug and water-tight.

With this done, the temporary inside "pudge-boards"

were removed, and the ragged edges of the damaged hull cut away by means of oxy-acetylene torches. In this manner all of the repairs were effected, and the total damage covered an area of 700 sq.ft. The "Royal George" left the St. Lawrence for Halifax, and on the way encountered heavy weather, but Mr. Wotherspoon's repairs proved quite equal to the stress placed upon them. At Halifax, because of the novelty of the work and to satisfy the insurance people, the liner was docked for a brief examination. Everything being found in excellent shape, the "Royal George" loaded with cargo and returned to England without further mishap.

Portable Asphalt Mixing Plant; Springfield, Mass.

The accompanying illustrations show a new type of semi-portable asphalt or bituminous mixing plant recently purchased by the City of Springfield, Mass., for pavement construction and repair. This plant apparently supplies a need frequently felt by pavement repair departments of cities for a relatively small compact mixing plant adaptable to various mixtures of asphaltic and bituminous concrete and sufficiently portable to avoid long hauls of the finished mixtures.

The capacity of the plant is claimed to be sufficient to turn out finished mixture for standard sheet asphalt pavement, 2 in. thick, at the rate of 75 sq.yd. per hour, or 750 sq.yd. per 10-hr. day. With coarser aggregates intended for bituminous macadam or asphaltic concrete, the capacity would be correspondingly greater.

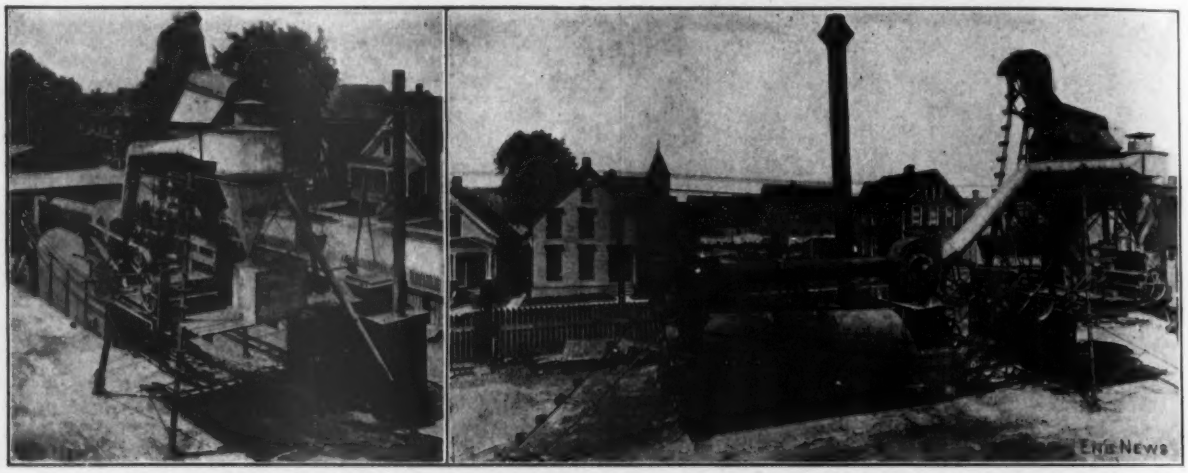
This capacity of plant is based on a drier capacity for handling graded sand, containing not more than 5% moisture, at a temperature of 77° F., and delivering it to conveyors at a temperature of 350° F. at the rate of 6 tons per hour; the mixer capacity is twice the drier capacity, so that by installing an additional drier and kettle unit the capacity of the plant may be practically doubled.

The plant consists of four units, a portable engine and boiler, a portable single-drum drier, a portable melting tank and a mixing tower, all of which are easily separated from one another, moved and re-assembled.

The boiler is of the 60-hp. locomotive type with a 55-hp. horizontal engine mounted on it. The whole unit is on four substantial iron wheels.

The drier consists of a single drum 40 in. in diameter and 20 ft. long, with firebrick and asbestos-lined steel setting, firing chamber, double rain-roof, bucket-elevator conveyors, etc. It is mounted on steel axles and wheels.

The melting tank or kettle has a capacity of 1556 gal. It has a refractory-lined fire unit, fire and ashpit doors, and is arranged for steam agitation, although the same outfit may be had for air agitation if preferred. The kettle unit is mounted on three 6-in. I-beams, so that it is readily handled by a derrick or crane. The mixing tower is made of four I-beam columns securely braced with tie rods and turnbuckles. To facilitate dismantling and reërection the several members are bolted together. The tower is equipped with a standard 9-cu.ft. steam-jacketed mixer, with two sets of shafts and teeth. The equipment of the mixer includes a two-compartment sand-bin having a capacity of 218 cu.ft., supported above between the legs of the tower. Directly over the sand-bin is a rotary screen and cradle for receiving the hot sand from the drier conveyors.



PORTABLE ASPHALT MIXING PLANT, SPRINGFIELD, MASS.

The tower is also equipped with a dust collector, so located that the dust collected is delivered above the top of the mixer, making it available for use as a filler. Accessories such as weighing box and scale for aggregates, träveler, bucket and scale for the bituminous material, power fan, link-belt elevators, etc., are parts of this unit.

The plant described was designed and built by the Barber Asphalt Paving Co., at its Iroquois Works.

A Stronger End Construction for Pullman Cars has been designed by Richmond Dean, General Manager of the Pullman Co. The fundamental element of the new design consists of two U-shaped I-beams placed beneath each car platform so that the legs extend upward through apertures in the platform casting and are firmly anchored to an end superstructure. Two adjacent legs of the I-beams form the door-posts at the entrance to the car while the others perform the same office for the opening between cars. These beams are made from one continuous I-beam instead of comprising riveted sections. This new car end is being applied to all new Pullman cars and to all high-class wooden cars as they pass through the shops for repair. The idea of the design is that the end shall act as a cushion to absorb and dissipate the energy of the impact in the event of a collision, and yet be economical as regards weight. For this information we are indebted to the "Railway Age Gazette," Aug. 8.

EDITORIALS

The following "discovery" by "chemists in the Department of Agriculture" has been appearing in the newspaper of late:

Although the ultra-violet rays may sterilize water in Europe, the vigorous bacteria in American milk refuse to succumb to their influences.

The scientists found that while the rays were efficacious in disposing of much [many] of the bacteria many of the pathologenic [!] disease producing germs survive.

In the course of the tests the Department chemists found it was impossible to sterilize completely even dirty milk bottles with the rays, let alone to accomplish the sterilization of the milk solids. Further inquiry as to the means adopted by the European scientists in the use of the rays probably will be made.

It hardly seems necessary to make experiments or to extend inquiries to Europe to find that the successful use of ultra violet rays has thus far demanded a clear liquid. It has been found necessary or desirable to clarify even slightly turbid water before applying the ultra-violet rays. With so opaque a substance as milk the use of this disinfecting agent appears to be so hopeless that one would hardly expect government scientists to attempt it.

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Clearness in Teaching Mechanics

We do not often give space in ENGINEERING NEWS to discussions of methods of teaching the different branches of engineering. The paper by Prof. Edward B. Huntington, however, abstracted in another column of this issue, seems to us well worth attention by the practicing engineer as well as by the teachers and students. Prof. Huntington's paper is a plea for clearness and simplicity in the teaching of mechanics and for the discarding of certain units used by physicists, such as the "dyne," the "erg" and the "poundal."

We venture the opinion that those who about a quarter century ago attempted to apply these units to the ordinary engineering student's work in mechanics performed a very bad service for the engineering profession. One of the most important qualifications of an engineer is the ability to think clearly and accurately; and this is one of the faculties that the engineering student should endeavor to acquire during his college course. It is because the introduction of these units has tended to confuse the thinking and reasoning of so many engineering students and engineers of the present generation that we say their application in this way was unwise.

And there is another reason why the use of such abstract scientific units is to be deprecated. Before an engineer gains many years' experience he finds that it is advantageous to dispense with most of his engineering nomenclature that he cannot make understandable by the average intelligent man. Engineering in its broad aspect is not a laboratory affair. It is the engineer who can make his plans and purposes intelligible to the average capitalist, power user, contractor, city council or railway magnate who is most apt to have his ideas and theories carried into effect.

It is a truism almost that the virile ideas which grip men's minds and influence them are always those expressed in concrete form and not as mere abstractions. Mathematical demonstrations may convince a few engineers; but a graphic illustration comprehensible by everyone will have a thousand times the influence upon a jury or a city council or a board of directors. It is all very well for the worker in the physical or chemical laboratory to use such physical units as will best suit his purposes and needs. The mistake comes when it is attempted to apply such units to the ordinary thinking and work of the ordinary engineering student and engineer.

⁂

A New School for Health Officers

The new Harvard-Technology School for Health Officers, outlined on another page of this issue, is notable in its aims and personnel, in its fine example of coöperation between two great educational institutions which some might expect to be rivals in this new field, and also in its joining and harmonizing of the medical and the engineering branches of public-health education.

In a country, which, until recently, has been almost without schools of public health, it is almost bewildering to be confronted with no less than 83 courses at the very inception of this project, but besides being in line with the ultra-specialization of the day it should be noted that the new school is designed to attract graduate students while at the same time making it readily possible for undergraduates of some training and capacity to qualify for the Certificate of Public Health (C. P. H.). Then, too, special students are invited, which also makes numerous courses desirable.

It is particularly gratifying to note that Bachelors of Science in Public Health and Biology are to be admitted on equal terms with Doctors of Medicine and that Graduates in Sanitary Engineering, or in Civil Engineering with certain specialization in sanitary engineering, will be allowed to try for a C. P. H. at the end of one year's residence. At the same time it should be noted that a few strictly medical topics, having only a remote relationship to actual public-health administration, are required for a C. P. H., while the "Requirements for Admission" close with the advice that candidates "obtain a medical degree before specializing in public-health work," since "preferment for position and advancement to the higher positions come more readily to those who have a medical degree." All this is a bit disappointing to those who have witnessed for years how much more real instruction in public-health administration has been given in a few engineering schools than in all the medical schools of the United States, and who have also witnessed the magnificent work done by a number of engineer health officers. However, it is a great advance to have the taking of a medical degree advised rather than compelled, and it is still true, though not so true as a few years ago, that laymen, generally, think M. D. is the only title which will warrant the appointment of a man as health officer or even as a member of a board of health.

Finally, Harvard University and the Massachusetts Institute of Technology deserve commendation for their coöperative enterprise, and the members of the Administrative Board of the new School of Public Health should likewise be commended for the broad and deep foundations which they have laid for the success of the school and for the liberal invitation which they have issued to graduates and students of medicine, engineering and related sciences to prepare themselves for efficient, modern health-protective work.

The Latest Garbage-Disposal Contract at Los Angeles

New York is not the only city which expects to sell its garbage for disposal by reduction (see ENG. NEWS, Aug. 7, 1913, pp. 267 and 269). As stated elsewhere in this issue, Los Angeles has recently awarded a contract under which it is to receive 51c. a ton for all the garbage which it collects and delivers to the contractor during the next ten years. We hope that Los Angeles will not repeat its unsatisfactory experience with two successive garbage furnaces, due largely to lack of competent engineering advice, if indeed any engineering advice was had.

Notwithstanding this experience the city authorities backed and filled, a year or so ago, over several propositions to have its garbage-disposal problem investigated, first by an experienced outside engineer, then by an engineer in the employ of the city and finally by a layman city official. In the end it invited bids on general specifications. It would not be at all surprising if Los Angeles soon has more complaints about the proposed garbage-reduction works than it has had about the great hog ranch to which the garbage of the city is now taken.

We do not mean to imply that the proposed reduction plant need be a nuisance nor that there was just cause for complaint about feeding garbage to 20,000 or more hogs. In fact, as to the hog ranch, a member of the editorial staff of this journal was much impressed with its good sanitary condition on visiting it about a year ago, and could not quite make out what real grounds the county authorities or anybody else had for insisting that Los Angeles stop getting free garbage disposal in that way.

All that seemed to be needed at the hog ranch a year ago to remove any reasonable claims of unsanitary conditions was the disinfection of the garbage by boiling in order to lessen the possible infection of the hogs by human tuberculosis; and we have seen no positive proofs that such treatment is necessary, though there are reasons for believing it to be desirable. It may be added that the privately owned Los Angeles garbage piggery is located miles from Los Angeles, in a sparsely settled area, most of which did not seem to be under cultivation.

Of course, if Los Angeles can change from free disposal to a revenue of 51c. per ton, with ample assurance that the new contract will be satisfactorily carried out, it will deserve congratulation, even if it is a matter of luck instead of good engineering, as seems to be the case. Unfortunately, there is nothing in the specifications that gives any such assurance, and the fact that the only competing bidder wanted to be paid 60c. per ton for garbage disposal (method not stated) suggests that Los Angeles may soon wish that its garbage was still being taken out of the city by rail and fed to hogs, without cost to the taxpayers. This is all the more possible because the garbage of Los Angeles appears to contain so large a percentage of vegetable matter as to make it better suited for hog food than for conversion into fertilizer. Obviously, this is a matter which should have been carefully studied before bids were invited; but in the matter of garbage disposal the Los Angeles way is to trust to luck and contractors, rather than to secure and follow the advice of engineers—as witness its two successive abandoned garbage furnaces, already mentioned.

Applicants for Engineering Positions on the Federal Railway Valuation Board

Ever since Congress passed the act requiring the physical valuation of the railways of the United States by the Interstate Commerce Commission, there has been much speculation amongst engineers as to just how the work would be carried out and what opening it would afford for engineers. The five leading positions were soon filled (see biographical sketches of the members of the Railway Valuation Board, June 15, 1913) and not long afterwards the Federal Civil Service Commission invited applications for a number of engineering positions (see p. 47 of our Construction News section for June 19, 1913). The applicants for the major positions were not called on to take examinations, but merely had to file papers showing their records. The other candidates were examined on July 21 and 23.

The positions to be filled commanded salaries of from $4800 to $1800. There were thousands of applicants. Although appointments will not be made for several weeks, we are able to give the following statement of the number of applicants for each class and grade:

	No. of Applicants	SALARIES Grade 1	Grade 2
Senior Structural Engineer	703	$3000 to $4800	$1800 to $2700
Structural Engineer	44	$1080 to $1500	
Senior Civil Engineer	3024	3000 to 4800	1800 to 2700
Civil Engineer	640	1200 to 1500	720 to 1080
Senior Inspector of Car Equipment	262	1800 to 3600	
Inspector of Car Equipment	295	1200 to 1500	
Senior Electrical Engineer	488	3000 to 4800	1800 to 2700
Electrical Engineer	132	1080 to 1500	
Senior Inspector of Motive Power	475	1800 to 3600	
Inspector of Motive Power	271	1200 to 1500	
Senior Railway Signal Engineer	146	3000 to 4800	1800 to 2700
Railway Signal Engineer	50	1080 to 1500	
Senior Mechanical Engineer	425	3000 to 4800	1800 to 2700
Mechanical Engineer	63	1080 to 1500	
Senior Architect	239	3000 to 4800	1080 to 1500
Architect	11	1080 to 1500	

Candidates for senior engineers in all classes and for senior architect, were required to have had five years of responsible, practical experience and to have graduated from a school of civil, mechanical or electrical engineering, or a school of architecture (according to the position); or if not a technical graduate, to have had ten years of professional experience. Candidates for the lower engineering positions (except that of structural engineer), were required to have had in some cases five and in others three "years of experience," toward which graduation from a technical school would count for three or for two years, respectively. All candidates required to take examinations were to be rated on the basis of 60 points for examinations and 40 points for education, training, experience, and fitness.

The relative number of applicants under the different classes affords an interesting subject for speculation from various points of view. The fact that 3024 men applied for the position of senior civil engineer, at salaries ranging from $1800 to $4800, seems to indicate either that many civil engineers are out of employment, or if employed would like to change to something which, if not more remunerative, is more certain to bring in a dependable sum every month. Presumably relatively few appointments will be made at $4800 a year in all the senior grades combined, but hope springs eternal in the human

breast, which fact, combined with the absence of examinations, goes far toward explaining why the senior positions brought out so many more applicants than the others.

The engineering profession will watch with the greatest interest, and no little concern, the announcements of the successful candidates. Unless the thousands of applications for those positions which did not require the candidates to appear for examination are judged by a high standard, presumably the eligible list will be a large one. This will give a fine chance to choose men on the basis of education, experience and general fitness. At the same time, it will open the door for possible favoritism. The valuation of the railways of the United States is a task so vast, complex and difficult, and so much of the economic, social and political welfare of the country depends upon its outcome, that it is sincerely to be hoped that nothing but fitness for the task will be given a thought in choosing men from among the thousands of candidates.

Lessons from a Never-Used Sewage-Treatment Plant

A few years ago a number of state boards or departments of health began to issue orders in a more or less wholesale way calling upon numerous cities and villages to install sewage-treatment plants. In several states these orders were based upon little or no investigation of local sanitary conditions. Generally no consideration whatever was given to the financial status of the municipalities involved. As a rule these orders emanated from physicians; well meaning men, perhaps, not themselves politicians but elevated to office by the chances of party politics, trained in private medicine but not well grounded in the principles of public health and sanitation, and quite ignorant of engineering costs and the financial limitations of cities. Such of these state health officers who were fortunate enough to have good engineering advisers, and wise enough to follow them, soon learned to temper with discretion their zeal to stop all discharge of "unpurified" sewage into the waters of their respective states, quite regardless of the relative volume of the sewage and the water and the uses to which the waters receiving the sewage were put. While these health officers were learning, some unfortunate orders were issued. Of such orders some were obeyed to the extent only of filing plans for sewage treatment. In other cases works were built but were never operated as designed. In one case, at least, the works built were never even put in operation.

Such evasions were in large part due to weaknesses in state legislation, the statutes requiring cities to obtain the approval of state departments of health before building or extending sewerage systems, but giving the department no authority to force the execution of the plans approved by them; or possibly some of these state departments assumed authority for requiring sewage treatment where no authority really existed. Later legislation has remedied some of these defects. In addition, the provision of competent engineering staffs, charged with the duty of investigating each sewage-disposal problem before a state order is issued, is fortunately becoming more common. As a result of improved legislation and the more extended use of competent engineering advice, it is to be hoped that fewer unreasonable orders for sewage treatment will be issued by state departments of health in the future than in the past and that the orders issued will be of such a character that, with the aid of proper legislation, they can and will be enforced.

The foregoing reflections have been prompted by the history of a sewage-disposal plant at Scotia, N. Y., which was brought to our attention by a Southern engineer traveling in the North. An investigation has brought out some interesting facts connected with this plant from which a few lessons may be drawn. Briefly the story is as follows:

Scotia is a village of some 3400 inhabitants located on the Mohawk River near Schenectady. Early in 1905 it submitted sewerage plans to the State Department of Health. Whether or not these plans included sewage-treatment works we are unable to state. Presumably they did not, for in April following plans for both sewerage and sewage disposal were submitted to the department and these later plans were approved subject to minor changes. In August of 1905 the modified plans were submitted to the department and approved by it. So far as appears no investigation as to the need of sewage disposal for Scotia was made.

In November, 1905, complaint was made to the State Department of Health that the village proposed to discharge all sewage from its sewerage system into the Mohawk River before the disposal works were built. An order was issued by the department forbidding this, but after a subsequent hearing, at which the village authorities brought out the fact that delays in the construction of the disposal works beyond their control had occurred, the commissioner of health modified the order forbidding the discharge of untreated sewage into the river.

This stipulation was really to the effect that if land for sewage-disposal works were acquired and a contract for the works let the commissioner would grant a temporary permit for the discharge into the river of untreated sewage from a limited number of houses, the permit to extend up to Sept. 1, 1906.

In April, 1906, the State Department of Health issued a permit in which it was stated that the conditions just named having been complied with, 250 houses might be connected with the sewers and the sewage discharged directly into the river without treatment till Oct. 1, 1906, when the permit for the discharge of untreated sewage would be automatically revoked.

Just what happened in the next two years or so does not clearly appear from the information which we have received. One report has it that the disposal works were actually completed in 1906, but another report states that they were not completed until 1908. Whichever may have been the correct date of completion there seems to be no question but what the works were never put in operation. We are informed that after two trial runs (either 1906 or 1908) it was found that the machinery (presumably the pumping plant) did not work satisfactorily and at the same time it was also found that the cost of operating the plant was going to be excessive. As a result the plant was never put in operation, although it is stated that in 1910 a new pump was substituted for the old one.

It was not until 1911 that amendment to the New York statutes relating to sewage-treatment works made

it possible for the Department of Public Health to compel municipalities to take sewage out of streams and this amendment made such a course possible only when it could be proved that the discharge of sewage was a nuisance or a menace to health. In the event of such evidence the order for sewage treatment issued by the Commissioner of Health must also have the approval of the Attorney-General and the Governor. Since this act was passed advantage of it has been taken in New York State to order a number of places to install sewage-treatment works. Included in these have been various municipalities on the Mohawk River. A recent investigation, however, has shown that no nuisance from sewage disposal exists on the Mohawk except at a few places. In view of this fact and of the further fact that all water-supplies from the Mohawk and Hudson Rivers except one are purified, it appears that the State Department of Health has no authority to compel sewage-disposal plants except at those places where nuisance exists.

In 1912 Scotia again brought itself under the direct supervision of the State Department of Health by asking approval for a sewer extension of a few hundred feet. This reopened the question as to its sewage-disposal works. The state authorities then officially learned that the sewage-disposal plant was not in operation on the alleged ground of the excessive cost which would be entailed by operation and the small amount of sewage being discharged into the river. Investigation on behalf of the State Department of Health showed that the plant was not economically laid out, the septic tank and the filter beds being located some distance from the pumping station with a dynamic lift of over 50 ft. between the station and the disposal works. The village plead for authority to make much-needed extensions to its collecting sewers without opening the sewage-disposal works—at least without opening them until such time as Schenectady with its population of 75,000, just below the Scotia sewer outlet, had installed the sewage-treatment works then being planned for the larger city. An investigation showed that no nuisance was being caused by the discharge of the Scotia sewage into the Mohawk River. Nevertheless, it seems probable that the State Department of Health would have ordered Scotia to put its septic tanks in operation had these been located so that the sewage could be discharged into them by gravity.

At the present time we understand that the village of Scotia has a permit revocable at any time to discharge raw sewage into the Mohawk River, but that it is the intention of the State Department of Health to compel the existing disposal works to be operated as soon as the Schenectady works are completed. This it can do if it sees fit, since the legislature of 1913 enlarged the powers of the State Department of Health in the matter of sewage disposal.

To sum up this story: Scotia was compelled in or about 1906 to spend some $16,000, according to reports, for a sewage-disposal plant, the operation of which would have cost it some $2000 a year. Difficulty having arisen in connection with the machinery and the cost of operation having been proved to be high, the village has let the plant remain idle for some five to seven years. The State Department of Health, which in the first instance either required the installation of this plant or else failed to save the village the necessary expense by suggesting that less extensive plant would be sufficient, was not for a number of years in a position to compel the plant to be put in operation. Moreover, the department after being equipped with a larger engineering staff than it had in the first instance, and perhaps because of experience gained in the meanwhile, appears to have seen the unreasonableness of its earlier order.

Although we, of course, have made no local investigation, it seems very probable that the most that could have reasonably been required of the village of Scotia in 1905 was sedimentation. It seems very likely that works for this sort of treatment might have been built near the river, thus avoiding a pumping main a mile in length, more or less, as well as the filter beds and possibly making unnecessary any pumping whatever. Even had filter beds been necessary later on, their construction might very likely have been postponed without harm to anyone wherever the disposal works might have been located. Obviously, if a site for the septic tanks could have been found near the river, pumping would also have been unnecessary or reduced in amount.

It is only fair to say that the engineering division of the State Department of Health has been reorganized and the personnel changed since the inception of the Scotia works, and in addition that the attitude of the department itself has become more in accord with a rational view of sewage disposal.

The lessons to be drawn from the Scotia incident are so obvious as to make it hardly necessary to state them, but four points may be indicated: (1) No state department of health or other central body controlling sewage disposal should order a community to install sewage-treatment works without first taking local conditions into account, including the volume and character of the stream into which the sewage is to be discharged, what is being done by other communities on the same stream in the way of sewage treatment, the financial condition of the municipality directly involved and the importance of its other sanitary needs as compared with the importance of sewage-treatment work. In so far as the sewage works for a given community are for the protection of non-residents, the principle of the greatest good to the greatest number and a broad economic aspect of the subject should be given careful consideration. (2) State departments of health should be vested with ample powers to enforce such orders for sewage disposal as they may give and also to make sure that any plans for sewage treatment which they approve are carried out—providing, of course, that the plans are rational ones, which it should be the duty of the state department of health to see is the case. (3) Furthermore, it is perfectly plain that if a state department of health is justified in insisting on sewage treatment and if it has authority to require that plans submitted to and approved by it be executed as shown then the department should also have power to compel the municipality involved to so operate the works when they are built as to give the results which the department in the first instance deemed to be necessary when it laid down the requirements as to sewage treatment. (4) Sewage treatment being largely an engineering question, the authority of the state to compel large outlays for sewage treatment and large yearly capital charges and operating expenses should be subject one way or another to the approval of a competent engineer, either as a responsible health officer or as engineering adviser to the health department.

LETTERS TO THE EDITOR

The Missouri Occupational Disease Law

Sir—Noting what you published (in ENGINEERING NEWS, Aug. 14) relative to the New York law governing compressed air, I thought that the occupational disease laws which are in effect in a number of states might be of interest to your readers.

Inclosed you will find a copy of the law enacted in Missouri, June 23, 1913. Laws substantially the same as this are in effect in the following states: New York, California, Wisconsin, Michigan, Connecticut, Illinois, New Jersey, Maryland, Maine, Minnesota, Ohio, Massachusetts, New Hampshire and Pennsylvania. The law was passed in most of these states this year, although in a few it has been in effect for some time.

W. J. REISS.

601 Frisco Bldg., St. Louis, Mo.
Aug. 16, 1913.

[The printed matter referred to by Mr. Reiss is a small pamphlet published by the State Bureau of Labor Statistics, Jefferson City, Mo. The law is defined as an act to promote the public health by protecting certain employees in Missouri from the dangers of occupational or industrial diseases, providing penalty, and providing for the enforcement thereof.—ED.]

A Warning to the Gullible

Sir—The general outline of Mr. Campbell's project to derive power from the different temperatures of sea water at different levels, as described in your issue of Aug. 7, is so extremely interesting that it may be well to consider some of the more unpleasant details before a lot of fakirs use the scheme as a means of extracting the savings from the bank accounts of poor widows and one-legged soldiers, as I have so often seen the inventors of various wave-motor schemes do on the coast of southern California where the crop of suckers never fails.

Having seen the great majority of these wave-motor schemes end in a pile of wreckage, I realize the almost impossibility of maintaining any floating structure exactly over a given point on the surface of the ocean, or connecting it to anything else in a mechanical way. Many of us well know the difficulty of keeping a dredger exactly where she belongs, even in comparatively smooth water, with steel-shod post-anchors and wire-cable guys. Consider then, the difficulty of keeping a floating power plant exactly centered over a 12-ft. flume, 1800 ft. long, during a south Pacific hurricane! And, if we succeeded in this, how long would it take her to ram herself to pieces on the upper end of it?

Again, is Mr. Campbell thoroughly familiar with the ravenous appetite of the limorea and the toredo for all wooden structures in sea water? Also, how would our old friend Kutter's "n" apply to a wooden flume, 1800 ft. long, after it had been coated with two feet of mussels and kelp 30 ft. long? If these surface growths did not flourish in the low temperature of water from the 1800-ft. level there are undoubtedly others that would.

Mr. Campbell's scheme includes, as one of the necessary features, a condenser. In a recent power-plant project engineered by the J. G. White Co., on the coast near Los Angeles, they spent approximately $30,000 to keep the "sea-cabbage," or fine-floating seaweeds out of the condensor tubes.

There are some sources of power which, though enormous in extent, never have paid for the difficulty of maintaining the necessary plant to use them. Gasoline engines have supplanted windmills. Countless wave-motors have gone to pieces before any dividends were forthcoming, and left steam engines operating on the beach; so, if Mr. Campbell's scheme is to work out finally, a careful look at the difficulties beforehand will hasten the achievement, and possibly block the unscrupulous promoters who would use it to operate on the gullible.

BENJAMIN BROOKS.

4241 Holmes St., Kansas City, Mo., Aug. 13, 1913.

An Oversight in the Compressed-Air Law

Sir—Working in compressed air is an occupation which has its dangers, and the conditions under which such work is carried on come legitimately under the cognizance of the law. The regulations prescribed in the amended law of the State of New York, very properly printed in ENGINEERING NEWS, Aug. 14, must have general approval, as to their intent at least.

As to the precise conditions to be insisted upon for compressed-air workers, and especially the time and pressure limits in decompression, there must always be more or less room for discussion, with differences of opinion unreconciled after all. When records and experiences are appealed to they are found to vary much in their testimony and to be more or less contradictory.

This may result perhaps from the overlooking of two important conditions having to do with the health and safety of compressed-air workers; viz., the breathable condition of the air, as determined principally by the volume furnished, and also its temperature.

It may fairly be assumed that the condition of the worker as he enters the decompression chamber must have its effect all through the decompression in determining its success and the time necessary for its safe completion. From discrepancies in the results of decompression, where the pressures and times have corresponded in the different cases, it may be a simple matter to assume that the condition of the subjects in the different cases as they have entered the decompression chamber (the conditions resulting from their hours of work under pressure and according to the character of the confined atmosphere, as to oxygen content, etc.) may not have been the same. If a man has worked his stent in the

compressed air with the air as pure and as rich in oxygen as possible, and also reasonably cool, he may be expected to be, all through the decompression and at the end of it, in much better condition than one who has worked almost to the verge of complete collapse in hot air which has not been constantly and copiously renewed.

There is nothing about the use of compressed air in caissons and tunnels which provides for any automatic renewal of the air, or the taking care of the condition of it in any way. The principal function of compressed air in the caisson is to prevent the water from forcing its way in under the lower edge of it, and, providing there were no leakage, the only compressed air functionally required would be for increasing the pressure as the caisson was sunk deeper. It is evident, however, that men could not live and work in air confined like that, and it therefore must be constantly renewed. It is only necessary to force in the air in sufficient quantity; the surplus will constantly escape under the lip of the caisson, and the pressure can never be increased above that required to resist the water pressure.

If sufficient compressor capacity is provided and kept operative it is all that can be required, but as this is an expense there is always some temptation to keep the supply of newly compressed air as low as possible, with the result that it may sometimes be deficient. This should not be permitted, and it would seem to be entirely within the scope of the law, when it undertakes to secure safe and proper working conditions, to insist that sufficient air shall be supplied and that the air within the working chamber shall be in as respirable condition as circumstances will permit. This would be to the interest of the contractor, however he might regard it, since it would enable the men to work with higher efficiency.

One objection to the use of the electric light in the caisson is that it gives no intimation of the condition of the air, as would a candle, and it might be very proper to require that at least one candle should be kept always burning in the caisson, and as long as its flame glowed bright and clear the men would have no occasion to worry about the respirable condition of the air.

There is, however, the temperature of the air to be considered. In caisson work, which for each individual caisson does not usually last long, it is customary to supply the air from a single-stage compressor located nearby. The law contemplates working in compressed air up to a limit of 50 lb. gage. When air with an intake temperature of 60° F. is compressed to only 20 lb. the final temperature is above 200° F., and with the intake temperature only a little higher the delivery temperature would be above that of boiling water. Some of this heat is undoubtedly lost in transmission, but the air must enter the caisson very hot, and it goes without saying that in every case an aftercooler should be required for the air, and that there should be a constant record of the temperature at which it enters the caisson, with a predetermined limit which should not be exceeded.

When compressed air is employed for the driving of subaqueous tunnels the condition of the air in the working chamber may be expected to be much better for the proper respiration of the workers than in the caisson. The air compressed is generally transmitted such a distance that the heat of compression disappears, so that little attention need be given to the working temperature, and then the air is constantly escaping, so that its renewal is a necessity for the approximate maintenance of the working pressure. At the tunnel face there is no balance of pressures as in the caisson. It must always be that in order to maintain sufficient pressure to hold back the water near the bottom of the tunnel there must be a considerable excess of pressure at the top of the tunnel and the air is bound to escape, demanding constant replacement.

When the tunnels were being driven under the East River for the Pennsylvania R.R., those who crossed the river by the 34th St. ferry had no need to be told how copious was the escape of the air, as the boiling of the water spoke for itself.

It is not difficult for us to believe that the excellent record of the East River tunnels as to caisson disease among its workers may be attributed, not only to the strict and careful decompression practice, but also to the excellent condition of the workers before submitting themselves to the decompression.

It will be noticed that the new law having to do with compressed-air practice deals almost entirely with the regulations for decompression, and this it does so thoroughly that the workers who submitted themselves to the decompression in strict accordance and in full compliance with the rules prescribed might be smothered in the chamber and taken out dead. Gradual decompression, which is the only thing insisted upon by the law, could be accomplished by merely shutting the men into the chamber under the full pressure and then gradually letting off the pressure according to schedule.

In the decompression, if nowhere else, the men should have pure air and lots of it, the air should be at a uniform temperature and the temperature should be that determined to be most suitable. Both these conditions are more or less imperative and are not difficult, although costing something, to provide, but the law says nothing about them.

FRANK RICHARDS.

Compressed Air Magazine, New York City,
Aug. 16, 1913.

Unit Stresses in Timber

Sir—The writer begs to differ with Mr. Dewell on some points made in his letter appearing in ENGINEERING NEWS, July 24, 1913, p. 179. The paper referred to in your editorial of June 26, 1913, p. 1335, discussed the waste of timber caused by the stresses specified in 29 building codes. Assuming 1800 lb. to be a rational fiber stress for long-leaf yellow pine the writer claims that the codes referred to require 15,990,241 people (census 1910) to consume enough lumber of this kind to serve the needs of 21,028,485 people; and that the expenditure of $539,375,184 for building operations in the cities considered required a use of this material sufficient for operations amounting to $724,619,580 during the year 1912; the first assumption showing a waste of 31.51% and the second a waste of 34.34%. I did not deem it necessary to state that this waste occurred in the use of structural timbers, as codes do not generally mention such matters as roof and wall sheathing, studding and flooring.

Building codes specify live-loads for various classes of buildings which are generally too high for residences, offices, apartments and hotels and involve a waste of material; they also specify fiber stresses for all kinds of

timber that are generally too low, which results in another waste of structural material. It was not the intention to give the amount of this waste in board feet of lumber or in money value, but to demonstrate the fact that there is a tremendous waste of a natural resource in the form of structural timbers of all kinds, caused by improper building code requirements.

The functions of a building code are limited to providing that structures be safe for occupancy, thus safeguarding life and property; that the housing, sanitation and means of egress conserve life and health; that the rights of adjoining property and the public are protected. Strength being a primary consideration, codes do not specify for deflection which is a factor of no consequence to the public, but is a matter for the consideration of the owner and his architects. One late code (Milwaukee) provides that beams be investigated for horizontal shear. No code that I have seen comprehends the consideration of the elastic limit of structural timbers, and the proper application of this physical property to designing is still an unsettled matter.

The fiber stress at elastic limit is given by different authorities as follows:

United States Department of Agriculture, Division of Forestry, Circular No. 12, p. 4:

	lb. per sq.in.
Longleaf yellow pine, air dry	8500
Shortleaf yellow pine, air dry	7200
Loblolly yellow pine, air dry	8150

U. S. Department of Agriculture, Forest Service, Circular No. 164, p. 14:*

	lb. per sq.in.
Longleaf yellow pine, partially air dry	4691
Shortleaf yellow pine, green	3322
Loblolly, partially air dry	3200
Loblolly, green	2500

*These values are 77% of those obtained by testing small sticks cut from the structural size sticks.

U. S. Department of Agriculture, Forest Service, Circular No. 213 for small, clear green specimens:

	lb. per sq.in.
Longleaf yellow pine	5090
Shortleaf yellow pine	4350

U. S. Department of Agriculture, Forest Service, Bulletin No. 108:*

	lb. per sq.in.
Longleaf yellow pine, green	3734
Longleaf yellow pine, air seasoned	3691
Shortleaf yellow pine, green	3237
Shortleaf yellow pine, air dry	4675
Loblolly yellow pine, green	3040
Loblolly yellow pine, air dry	3517

*These values for green material are 75% of those obtained by testing small sticks cut from them, and for air-dry material the average is about 60%.

University of Illinois, Engineering Experiment Station, Bulletin No. 41:*

	lb. per sq.in.
Longleaf yellow pine, average of Series A and B	4359
Shortleaf yellow pine	4343
Loblolly yellow pine, average of series C and D	3227

*The ratio of this stress developed in small specimens is for longleaf 66% and for loblolly (series D) the same.

The above values are for structural sizes unless otherwise stated and a careful study of the description of each piece develops the fact that many of them were not of a "merchantable" quality. Much higher averages would be obtained by testing timbers graded by the standard grading rules.

Mr. Dewell is mistaken in assuming that I advocated a factor of safety of six—the paper referred to reads as follows: "In the light of modern scientific investigation into the physical properties of woods there is no excuse for the old factors of safety of six or more, which were and are yet applied to woods." Six is truly a "factor of ignorance" and has no place today in building construction of any kind.

As long as the laws require that designing be based on an extreme fiber stress there is no other alternative and a stress of 1800 lb. for long-leaf yellow pine would be on a basis of a factor of safety of four corresponding to the modulus of rupture and a factor of safety of two and one-half to three corresponding to the elastic limit. Conditions do not warrant any greater margin of safety.

Referring to Mr. Dewell's reference to the actual size of dressed timbers I will say that the Yellow Pine Manufacturers' Association has in press a handbook containing tables based on these actual sizes with complete tables of the properties of such timbers. This handbook will be sent, free of charge, to those requesting copies.

In October, 1895, the Association of Railway Superintendents of Bridges and Buildings adopted a table of working unit stresses for railroad bridges and trestles. The name of this society was later changed to the American Railway Bridge and Building Association and the working stresses recommended in 1895 have never been revised. In March, 1909, the American Railway Engineering and Maintenance of Way Association adopted a schedule of working unit stresses for railroad bridges and trestles. The name of this association was later changed to the American Railway Engineering Association. Being two separate and distinct associations one did not revise the work of the other as stated by Mr. Dewell.

The working unit stress adopted in 1895 for long-leaf yellow pine is 1200 lb. and for short-leaf yellow pine 1000 lb. That adopted in 1909 for long-leaf yellow pine is 1300 lb. and for short-leaf yellow pine is 1100 lb., with the recommendation that for highway bridges and trestles the stresses be increased 25% and for buildings and similar structures in which the timber is protected from the weather and practically free from impact the stresses be increased 50%. When the 1909 schedule was adopted, Prof. Jacoby said: "We all remember that when the excellent report of 1895 was made by a Committee of the American Association of Railway Superintendents of Bridges and Buildings, of which the late Mr. Berg was chairman, that the part of the report on unit stresses in the form of two tables was copied very widely, and in many cases the results were used in a different way from that intended by the Committee."

As some engineers and text-book writers applied railroad bridge and trestle stresses to buildings the increased stresses for highway bridge and building work was recommended. It is incomprehensible that an intelligent engineer does use the same stresses for a building that are safely applied in designing railroad bridges and trestles, and yet many of our building codes require this to be done and some engineers offer excuses for such practice.

For 25 years Kidder's "Architects' and Builders' Pocket-Book" has been used throughout this country and some codes have adopted its recommendations. The tables in this book for yellow pine timbers are based on an 1800-lb. stress resulting in untold millions of feet of such material being so used with satisfactory results. It is the lawful fiber stress for a population of 2,640,178 people and the buildings in these cities are as safe, substantial and satisfactory as can be found anywhere in this country. Will Mr. Dewell cite instances where buildings designed for this stress have failed to satisfactorily fulfill all requirements?

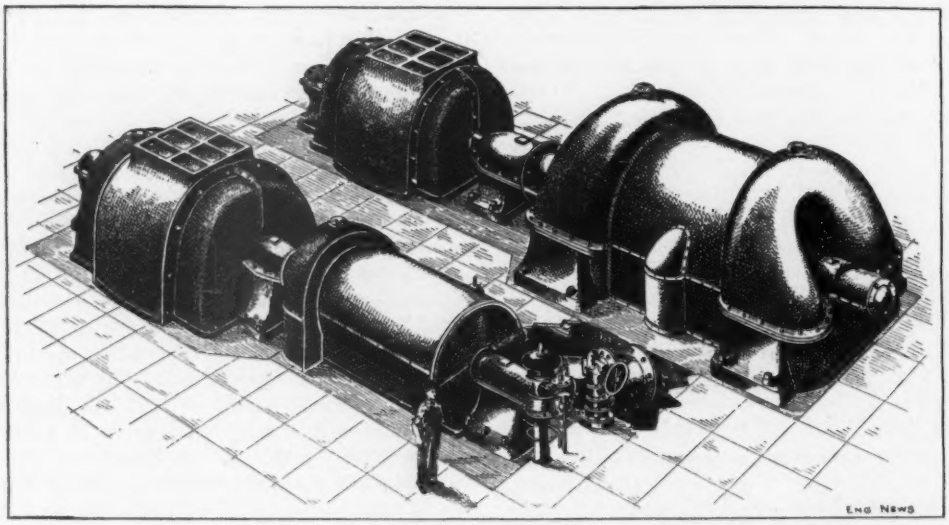
ARTHUR T. NORTH,
Consulting Engineer, Yellow Pine Manufacturers' Association.
Chicago, Aug. 6, 1913.

The Interborough Rapid Transit Company's 30,000-Kw. Steam Turbine Generating Units*

The awarding of a contract by the Interborough Rapid Transit Co., of New York City, to the Westinghouse Machine and Westinghouse Electric & Manufacturing companies for three 30,000-kw. steam-turbine generating units, was noted in ENGINEERING NEWS, Aug. 14, 1913, p. 324.

The general features of design are shown in the accompanying figure. The problem in this case was to design turbines of the very highest obtainable efficiency, nearly regardless of cost—the only more important consideration being reliability of operation. For want of a better term, the new type has been called "cross compound."

Turbines of this capacity can be designed in single units, operating at 750 r.p.m. Such machines would be relatively economical and would probably show a steam-consumption performance higher than has hitherto been obtained. However, the turbine-cylinder structure, on account of the slow rotative speed, would be relatively large and this, together with the temperature differences existing within the one structure in a machine of such large capacity, would make its construction an engineering problem of some magnitude. On the other hand, both steam turbine and generator of this capacity might be designed and constructed to operate at 1500 r.p.m. In this case, the structure would be less gigantic. But in order to avoid congestion of the steam in the low-pressure portions of the turbine and efficiently to expand down to the low limits of condenser pressure—29-in vacuum in this instance, blade speeds would be involved rather beyond what have hitherto been considered the limit of good practice. Either of these would be a combination-type machine, comprising an impulse element for the first expansion, followed by an appropriate number of reaction elements for the low-pressure stages, the latter being arranged for double flow. However, the highest degree of economy, according to the Westinghouse designers, is not to be obtained with an impulse element, as compared with a reaction element, providing the steam volumes, speed, etc., can be made appropriate for the best design of reaction turbine. In any turbine designed for high expansion ratios, the engineer is confronted with the problem of having to deal with relatively minute volumes of steam at the high-pressure end, and enormous volumes at the low-pressure, the volume of increase being roughly in the order of a geometrical progression. Therefore, if the high-pressure portions of a turbine may be operated at twice the rotative speed of the low pressure, the dilemma is largely eliminated. The capacities and speeds in this particular application, that is, the high-pressure half being a single-flow turbine operating at 1500 r.p.m., and the low-pressure half a double-flow type operating at 750, render it possible to design reaction machines which will have an efficiency beyond anything yet heard of. The blade speeds involved are low, and either turbine element is of exceedingly simple mechanical construction, involving no engineering problems whatever and thoroughly fulfilling the primary desideratum of absolute reliability.



PERSPECTIVE VIEW OF A 30,000-KW. "CROSS-COMPOUND" STEAM-TURBINE GENERATING UNIT

The scheme of employing two separate turbine elements, with the steam passing serially through them is not new. A number of such units, of from 1000 to 2000-kw. capacity, were built by the Westinghouse company in 1901. This construction has again come to the front in the case of the English-built Parsons turbines purchased by, but not yet delivered to, the Commonwealth Edison Company of Chicago (see ENGINEERING NEWS, Apr. 12, 1912, p. 750). It is new, however, to employ high-pressure and low-pressure elements, driving separate generators, each at a different synchronous speed.

There are other advantages in dividing such large turbines into two separate elements, besides those arising from the different speeds: the temperature range in either element is reduced; a sufficient number of stages may be introduced to give the highest thermodynamic efficiency without any mechanical difficulties, such as in-

*From a statement by H. A. Rapelye, Commercial Engineer, Westinghouse Machine Co., East Pittsburgh, Penn.

creased length between bearings, etc.; some little advantage is to be gained by separating out, between the two turbine elements, the water which has been precipitated in the first expansion; either element is more reliable and simple, because of its smaller size.

Doubtless, there is some slight loss of efficiency due to the employment of two generators, instead of one of twice the capacity. However, this is more than overbalanced by the gain in economy due to the two elements operating at different speeds.

The guarantees of steam consumption on these machines for the Interborough Rapid Transit Co., are considerably better than those on the English-built Commonwealth machines. The pounds of steam per kilowatt-hour guaranteed in each case are approximately the same, but the operating conditions are much less favorable for the Interborough units—the superheat being 200° as against the 120° contemplated for the Interborough plant. The cold waters of Lake Michigan will enable 29.2-in. vacuum to be used instead of the expected 29-in. at New York. The guarantees for the Interborough turbines provide for 75¾% of the energy available in the expanding steam converted into electrical energy at the terminals of the generators. This, roughly, is a 7½% better guarantee than that reported for the Chicago machines.

Selling Garbage for Reduction at Los Angeles

By E. Allen Phillips*

Buying garbage from the city and making money out of it by the reduction process is the enterprise which C. D. Crouch has undertaken in Los Angeles, Calif. So confident is Mr. Crouch of his ability to boil, cook, squeeze and otherwise cajole gold dollars out of ordinary household refuse, that he has offered Los Angeles 51c. per ton for all it can produce in the next ten years. In addition he put up a bond of $10,000 with his bid as assurance that he would enter into a contract if awarded to him. When the contract is executed he will be required to put up a $50,000 bond for its faithful performance. The contract is now (Aug. 12, 1913) being prepared and will be executed soon. It will be based on the specifications as advertised and on the bid of Mr. Crouch in which he offered:

To dispose of by the process of reduction all garbage produced and to pay to the city 51c. a ton therefor.

To dispose of by the process of reduction all dead animals, including ox, cow, steer, horse, mule or other draft animal, and pay to the city for these at the rate of $3 each, and for calf, sheep, hog, goat or animal of similar size—not including dogs and cats—80c. each; it being understood that the contractor is to receive the compensation for collecting the dead animals that the city now pays.

To dispose of dead dogs and cats by reduction without cost to the city.

To dispose of all market refuse by reduction without cost to the city.

To dispose of combustible rubbish by incineration without cost to the city; it being understood that the contractor will be allowed to charge the collectors of this rubbish the same fee now charged them by the city for such disposal.

To erect and operate a plant at his own cost and expense at a point within the city to be mutually agreed upon; if plant is located outside the city limits and requires transportation of the garbage, then the waste is to be delivered f.o.b. cars, Los Angeles.

*Room 14, City Hall, Los Angeles, Calif.

The above are the principal points to be covered in the contract, the bidder reserving on his part the exclusive right to the disposal of all the garbage and dead animals, but leaving it optional with the city to at any time take care of the dogs and cats, market refuse and combustible rubbish.

Market refuse is defined as the spoiled or condemned fruit or vegetables and similar waste from the wholesale and business section of the city and from the public markets. This is now collected by private parties and hauled to an incinerator where the city destroys it, charging 6c. per 100 lb. for so doing. Combustible refuse is handled in the same manner except that it is destroyed by the city for 3c. per 100 lb.

All the provisions of the contract will contemplate the delivery of the garbage by the city to the contractor's plant. This is because the city attends to its own collections by teams, a condition which was forced upon it by the failure of a former contractor to carry out the terms of his agreement. Collections of garbage in Los Angeles total from 125 to 135 tons daily as a rule. In the melon and sweet-corn season, this is increased 50 tons a day. It costs the city approximately $85,000 a year to make the collections and it is figured that with the new contract this expense will be offset by about $25,000, the revenue from selling the garbage to the new disposal contractor.

All that was required of the bidder was to specify a "reduction plant" and Mr. Crouch has never officially vouchsafed any information as to the sort of a plant he intends to erect.

Los Angeles garbage now is disposed of on a ranch on which are at present something like 28,000 hogs. The contract for disposing of the garbage in this manner—which, by the way, is without compensation to the city—expires in about six months. Some arrangement probably will be made for changing from one system to another by degrees so as not to work a hardship on the hog owners by forcing them to dispose of the animals hastily.

Three prime causes resulted in a determination on the part of city officials to change the system of garbage disposal:

(1) Objection of outside communities to the maintenance of hog farms in their vicinity.

(2) Public sentiment of people inside the city against garbage-fed pork, created largely by insanitary farms maintained near the city a few years ago.

(3) A desire among certain city officials to obtain a revenue from the disposal of garbage.

The outside communities were the ones to finally bring the question to an issue. Although the hog ranch is exceptionally well located in the dry sandy bed of a river about 15 miles from the city, the sparsely settled communities in the vicinity of the 28,000 swine succeeded in bringing pressure, political and otherwise, to bear on the county authorities to such an extent that the Board of supervisors passed an ordinance forbidding the disposal of garbage anywhere in Los Angeles county outside of an incorporated city. This ordinance was passed more than a year ago, but has been held in abeyance upon the promise of the city officials to provide some other method of disposal. This ordinance the legal department of the city was willing to contest in the courts and assurance was given that it could not legally be enforced, but public sentiment against this method of disposal, together

with the risks attendant upon a battle in the courts, resulted in a promise by the city to find some other method of solving the problem.*

For some time prior to three years ago, the city's garbage was collected in an unsatisfactory manner by a private contractor and hauled with teams to hog farms maintained by private parties at the very boundaries of the city. But little care was taken to observe ordinary rules of sanitation, either in the collection of the garbage or the conduct of the farms, and conditions became so bad that they undoubtedly disgusted many persons with this method of disposal. When the city took over the garbage-collection contract, the different hog raisers combined and installed the one large ranch above alluded to and the city erected a transfer station and kept it in sanitary condition, where the refuse is transferred from wagons to railway freight cars and hauled to the ranch.

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The Harvard-Technology School for Health Officers

A notable plan for coöperation between Harvard University and the Massachusetts Institute of Technology in an important field of teaching will go into effect with the opening of a School for Health Officers on Sept. 29, 1913. The school will be governed by an administrative board of three members, appointed by the corporations of the two institutions. The board organized on July 30, as follows: Chairman, Dr. William T. Sedgwick, Professor of Biology and Public Health, Massachusetts Institute of Technology, 30 Trinity Place, Boston; Secretary, George C. Whipple, M. Am. Soc. C. E., Professor of Sanitary Engineering, Harvard University, Cambridge; Director of the School, Dr. Milton J. Rosenau, Professor of Preventive Medicine and Hygiene, Harvard Medical School, 240 Longwood Ave., Boston.

Each member of the board is well known for his long service in the field of public health and sanitation. Professor Sedgwick, besides his long teaching career, in which he has instructed and inspired many of the leading health officers and sanitarians of the country, including Professor Whipple, was for some years Consulting Biologist to the Massachusetts State Board of Health and has made a notable record as an epidemiologist. Dr. Rosenau was long connected with the U. S. Public Health and Marine Hospital Service and was director of the Hygienic Laboratories of the Service when he went to Harvard a few years ago. Professor Whipple was for many years Biologist for the Boston Water Board, then Director of the Mount Prospect Laboratory of the New York Water-Works. For some years past he has been, as he still is, a member of the engineering firm of Hazen & Whipple, New York City. Each member of the board is author of important books on public health and sanitation.

Besides the men on the administrative board the teaching staff of the new school will number over forty. Most of these are connected with one or the other coöperating institutions, including Prof. Dwight Porter, of the Institute, who will deal with hydraulic and sanitary engineering, and Prof. Theobald Smith of Harvard, who will lecture on the relation of animal diseases to the public health. Dr. Mark W. Richardson, Secretary of the Massachusetts State Board of Health, will lecture on practical health administration and Dr. C. V. Chapin, superintendent of health, Providence, R. I., on municipal sanitation.

The new school will offer 83 courses, divided into the nine groups: Preventive Medicine, 10 courses; Personal Hygiene, 12; Public Health Administration, 8; Sanitary Biology and Sanitary Chemistry, 19; Special Pathology, 3; Communicable Diseases, 6; Sanitary Engineering, 11; Demography, 3; Medical and other sciences, 11 courses.

The school will award a certificate of Public Health (C. P. H.) signed by the authorities of Harvard and the Institute, to candidates who satisfactorily complete their studies in an approved schedule during not less than one year of academic residence. Schedules may be selected to meet individual needs but must cover in a broad way the knowledge requisite for the varied duties of a public health officer. Required subjects are: anatomy, physiology, pathology, biological chemistry, sanitary biology, preventive medicine and hygiene, demography, and sanitary engineering. It is expected that students will include courses in both of the coöperating institutions. The following official statement, covering requirements for admission and for a certificate, shows what credits are given for work previously done at the two institutions or elsewhere:

Graduates in medicine of Harvard University and other recognized medical schools will be admitted to the school for Health Officers upon their records and registered as candidates for the Certificate of Public Health. Bachelors of Science in Biology and Public Health of the Massachusetts Institute of Technology and other recognized institutions will likewise be admitted and registered as candidates for the certificate.

Masters of Civil Engineering of Harvard University who have specialized in Sanitary Engineering and Bachelors of Science in Sanitary Engineering of the Massachusetts Institute of Technology and other recognized institutions who lack the necessary preparation in medical and other sciences, will be admitted to the School upon their records, but will be required to spend at least one year in preparation before being accepted as candidates for the Certificate of Public Health.

Other graduates of colleges or technical or scientific schools will be admitted to the School without examination provided their collegiate courses have included adequate instruction in physics, chemistry, biology, French and German; but they will be required to spend two or more years in preparation before being accepted as candidates for the Certificate of Public Health. Applications for admission to the School will be considered from those who have spent at least two years in a recognized college or technical or scientific school and have pursued satisfactory courses in physics, chemistry, biology, French and German, and also from persons of unusual experience or special qualifications, but they will be required to spend two or more years in preparation before being admitted as candidates for the certificate.

Special students not candidates for the Certificate of Public Health who desire to fit themselves for some special field will be admitted to the School and may take any course or courses for which they are properly qualified on approval of the Administrative Board.

While the medical degree is not a prerequisite for the Certificate of Public Health, candidates are advised to obtain the medical degree before specializing in public health work. Experience teaches that preferment for position and advancement to the higher offices come more readily to those who have a medical degree.

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Untreated Hemlock and Tamarack Railway Ties decayed after 5½ years' service in a test conducted near Jonesville, Wis., while treated ties were "practically as good" at the end as at the beginning of the period.—Forest Notes, U. S. Forest Service.

*The city, nevertheless, asked for bids for garbage disposal by feeding to hogs, as well as by reduction and incineration. Besides the bid here reported one was made for disposal at a plant within the city (method not stated), but 0cc. per ton was demanded of the city.—Ed.

The Pennsylvania Public-Service Law

What is officially designated as the "Public Service Company Law" passed by the last session of the legislature of the Commonwealth of Pennsylvania was approved on July 26 by Governor Tener. This law, in addition to establishing a regulating commission having jurisdiction over public-utility services and businesses, provides for a limited control over municipal-utility departments, for the control of railway (and other utility) crossings, for the abolishing of the old Railroad Commission and the repeal of conflicting legislation. Some of the more important provisions of the law are summarized below:

The act starts out by defining a "public service company" so as to include all the usual utilities and common carriers and also stage-line, express, baggage-transfer, pipe-line, ferry, sleeping-car, dining-car, tunnel, turnpike, bridge, wharf, inclined-plane, grain, elevator, telegraph and telephone, natural-gas, heating, refrigerating, and sewage corporations, and all persons engaged for profit in any such business. "Service" is defined so as to include the interchange of facilities between two or more companies.

The law, in stating the duties of public-service companies, requires them first to furnish, without discrimination, safe and reasonably adequate service at reasonable rates. All tariffs, schedules, rules, contracts, leases, reports, etc., have to be filed with the commission. No changes in any tariff or schedule are to be made, except after 30 days' notice to the commission and to the public, unless the commission consents to a shorter time. Rates once fixed by the commission are not to be changed for three years without approval.

A further imposed duty is the adoption and use of uniform systems of accounts and records, such as may be prescribed by the commission, carrying no charges in any operating account which can properly be classed as capital, and vice versa, and carrying proper and reasonable depreciation as may be required. The imposition of uniform accounts, depreciation reserves, etc., is laid also on all municipal corporations with respect to their public-utility departments.

The duty is laid on the companies of furnishing the commission all maps, profiles, engineers' reports, etc., of any aid in valuation of property, and of coöperating with the commission in the valuation. These companies are obliged to account to the commission, when required, for the disposition and application of the proceeds of all sales or pledges of stock, bonds, or other evidences of indebtedness, and to apply such proceeds in accordance with reasonable rules and regulations which the commission may impose.

Railroads are to construct, maintain and operate switching connections at reasonable places and on reasonable terms where the business offered justifies the cost. The conveyance of passengers or freight by common carriers and the transmission of communications by telephone and telegraph companies is to be continuous, without unreasonable interruptions or delays. Common carriers receiving property for transportation are liable for damage over both their own and connecting lines, but are empowered to collect from the company on which the damage actually occurred. Railroad and street-railway companies, if required by the commission, must construct and maintain connections between lines of the same character so as to form a continuous line of transportation and to establish through routes, through service and through rates. But no such company is required to give the use of its tracks or terminal facilities to any other common carrier. Telephone and telegraph companies in like manner are required to connect their lines for through service in the transmission of communications.

The powers and limitations of companies as described by the law, make it lawful to classify services and rates according to the nature, quantity and time of service, as well as the bulk, value and facility in handling of commodities. It is lawful to establish approved "sliding scales" automatically raising dividends (above the basic reasonable return) as rates are reduced, in order that the company may participate in economies, efficiencies and improvements in methods of service yielding additional profits. Advance payments, minimum payments, deposits for security, discounts for prompt payment, and penalties for failure to pay promptly are permitted if applied without discrimination.

All schemes are declared illegal, except under the special authorization of the commission, for securing greater compensation in the aggregate for the conveyance of passengers or property for shorter than for longer distances over the same line in the same direction, where the shorter is included in the longer distance. A similar provision applies to the transmission of communications by telephone and telegraph concerns.

All future public-service companies can be incorporated only after securing from the commission a "certificate of public convenience"; only likewise may existing public-service companies renew or supplement their charters or obtain additional rights, powers and privileges. Outside companies may exercise their powers and franchises within the state with the approval and certification of the commission. The sale, assignments and transfer of property, powers, franchises, etc., can be made only with such permission.

A municipal corporation may acquire, construct or operate any public-service plant or facility only on approval of the commission. But any municipal corporation, now operating, may continue or may extend its operations within its territory without the consent of the commission, and any works under construction may be completed and put into operation without approval.

It is made lawful for public-service companies to issue stocks, bonds and other evidences of indebtedness only for money, labor, or property; and the commission's "certificate of valuation" may be secured to show that this section has been complied with. In instances where a certificate of valuation is not sought a public-service company increasing its obligations is required to file with the commission on or prior to the date of issue, a "certificate of notification" showing the total amount of securities to be issued, the amount of outstanding prior issues, the amount theretofore required, the amount theretofore disposed of or held in the treasury (as a free asset or pledge), the amount of the new issue and its purpose, with full details of sale, pledge, par value, preferences and privileges, rate of interest, time of maturity, redemption value, etc. But the requirements of the act in regard to certificates of valuation or notification do not apply to issues payable within 12 months.

A railroad or street railway may cross a public highway, or the tracks of another company, at, above, or below grade, as directed by the commission. The facilities of one utility may cross similarly a highway or other utility works.

It is made unlawful to capitalize franchises and privileges in excess of their cost, or leases and contracts for sale or consolidation; issues of securities are illegal in substitutions exceeding the aggregate value of properties consolidated, plus any additional value actually contributed as determined by the commission. It is made unlawful to acquire, or hold a controlling interest in any public-service company doing business within the state without the consent and approval of the commission, but a minor interest may be held by one company in another subject to existing laws, without the approval of the commission. This requirement does not affect the control of corporations heretofore legally held, or diminish the previously acquired rights of any company in another; it does not prevent the future acquisition of stocks, bonds and other evidences of indebtedness where the major interest was acquired prior to the date when this act became effective.

Contracts or agreements between a public-service company and a municipal corporation are not valid unless approved by the commission; on notice to the local authorities, a public-service company may apply to the commission before securing consent of the local authorities for a declaration of the terms and conditions on which it will grant approval of contracts or agreements.

This law establishes "The Public Service Commission of the Commonwealth of Pennsylvania," consisting of seven members appointed by the governor with the consent of the senate. The commissioners as at first appointed, are to hold office for four, five, six, seven, eight, nine and ten years respectively, from July 1, 1913, and until their respective successors have been duly appointed and qualified, these successors being appointed for terms of ten years each. A quorum is secured with four members who must act unanimously for all purposes. Any investigation, inquiry or hearing within the power of the commission may be undertaken before or by any one of the commissioners when properly authorized to act for the whole body, but any determinations, rulings or orders of such a commissioner are not to become effective until approved by a quorum. The commission appoints its secretary, a marshal, an investigator of accidents, and such officers, experts, engineers, statisticians, accountants, inspectors, clerks and other employees as it may deem necessary. The general counsel of the commission is "ex officio" the state attorney general; he is directed to appoint two attorneys as counsel and assistant counsel for the commission.

The salaries of the commissioners are fixed at $10,000 each except in the case of the chairman, who is to receive $10,500. The secretary is to receive $5000, the counsel $7500, the assist-

ant counsel $5000, the investigator of accidents $5000, and the marshall $2000. The salaries of all other officers and agents and employees are to be fixed by the commission.

The proposed powers and duties of the commission are defined in great detail and in harmony with the various provisions of the act already noted. More particularly, also, it is given power to act on complaint or on its own motion. It may ascertain the fair value of the property of every public-service company in the commonwealth, exercising this power whenever required for any of the provisions of the act. The commission is permitted, in determining fair value, to consider every pertinent fact, including original cost—particularly with reference to amounts expended in existing improvements, market value of bonds and stocks, probable earning capacity of the property under any particular rates and with expenditures for obsolete equipment and construction as warranted by circumstances or the development of the enterprise, reproduction cost of property, development and going-concern value. All of these are to be given only such weight as is just and right.

The commission has power to subpoena witnesses, to compel testimony and the production of records. It may withhold from the public any facts or information obtained during any investigations if such action is in the interest of the public. All hearings are to be public. No individual can be excused from testifying or producing records on grounds of incrimination, but immunity from prosecution is extended.

Petitions for rehearing may be made within 15 days after the service of an order and the commission may grant one if it judges good cause shown; but an application for rehearing is not to operate as a supersedeas or to stay or postpone the enforcement of an original order, except as the commission may direct. Any party to a proceeding may appeal from the finding of the commission within 30 days to the Court of Common Pleas of Dauphin County (in which the capital is located), which court is clothed with exclusive jurisdiction throughout the state for hearing such appeals. The court by an interlocutory order may make such appeal act as a supersedeas after notice to the commission and hearing. The orders of the commission are to be taken as prima facia evidence of the reasonableness thereof, and the burden of proving the contrary is upon the appellant. No new evidence can be received at an appeal hearing, but if the court is satisfied that new evidence has been discovered, it may remand the case to the commission to take such new evidence, and act in accordance

If the court finds an order unreasonable, based on incompetent evidence, or not in conformity with law, it may reverse the order of the commission or may remand it with instructions to reconsider

An Electric Hydraulic Dredge

SYNOPSIS—The use of electric power for operating dredging plants is often advantageous and economical, the current being either taken from an existing station or operated at a special station where the work is sufficiently extensive to warrant it. The accompanying article describes an electrically operated suction dredge, the current for which is purchased from an electric railway in the vicinity of the work.

An extensive canal and levee system is being constructed for the protection of the city of East St. Louis, Ill., and the surrounding district from floods due to high water in the Mississippi River or in the streams flowing across the low-lying bottom lands to the river.* This is being carried out by the East Side Levee and Drainage District, the Chief Engineer of which is T. N. Jacob, of East St. Louis, Ill.

That part of the work which lies along the Mississippi River includes the levee proper, together with an extensive fill behind and in front (partly for the reclamation of low land), and piling and mattress work for its protection. The levee and fill, comprising some 6,000,000 cu.yd., are being built of sand, of which the river bed furnishes an unlimited supply. This part of the work was sublet by the Illinois Dredging & Construction Co. to the Kinser Construction Co., of St. Louis, Mo. The latter firm decided to employ an electrically operated hydraulic dredge, and has furnished us with particulars of the equipment and performance of this interesting machine.

The crest of the levee is to be built to a height of 45 ft. above zero stage of the river, and the maximum distance any part of the material for the fill behind the levee must be pumped is about 2700 ft. from the inner line of available material in the river. As the dredge is so constructed that it can take a 30-ft. cut in the bottom of the river with a depth of water of 40 ft., by taking advantage of high stages of the river, the maximum duty required of the dredge is to pump through 2700 ft. of pipe line against a 25-ft. direct lift.

*"Engineering News," Nov. 28, 1907, and Feb. 2, 1911.

To meet these requirements, the Kinser Construction Co. had the Morris Machine Works, of Baldwinsville, N. Y., design and build the equipment for an electric hydraulic dredge. This is named the "St. Louis" and is shown in Fig. 1. The hull is of wood 130x35 ft., and 9 ft. deep. The machinery equipment consists of a 20-in. manganese-steel pump, with interchangeable runners of 54-, 58- and 60-in. diameter, direct-connected to a 1000-hp., 2200-v., 5-cycle, 365-r.p.m. induction motor, capable of operating continuously at 25% overload. This pumping plant is shown in Fig. 2.

The motor is of the slip-ring type with drum controller and external starting resistance. In addition to this, the motor is furnished with external rotor resistors and suitable switches, so that 15% continuous speed reduction may be obtained if desired. Because of the practically constant-speed characteristics of the induction motor it was necessary to use different-sized runners for

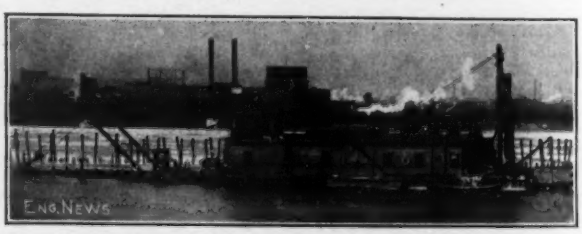

FIG. 1. ELECTRICALLY OPERATED HYDRAULIC DREDGE FOR BUILDING LEVEES AT EAST ST. LOUIS, ILL.

wide variations in lengths of pipe line and lift, the 54-in. runner being used on short lines and the 60-in. runner on the longest lines.

A steel ladder at the bow (guyed from a head frame) supports the suction-pipe, cutter-head, driving-shaft and gearing. The ladder is 80 ft. long and is pivoted on two hollow pivots. Through the one on the right passes the suction-pipe, and through the one on the opposite side the cutter-shaft is driven by a countershaft geared to a 75-hp. slip-ring induction motor. In the inboard suction pipe is a sluice provided with heavy bars to prevent large

boulders from passing through to clog or break the pump. The pump is located at about the center of the hull transversely to its length. The discharge pipe passes back on the left side to the stern, connecting with the pontoon pipe line.

For handling the dredge there is a seven-drum Lidgerwood hoisting equipment, driven by a 75-hp. slip-ring in-

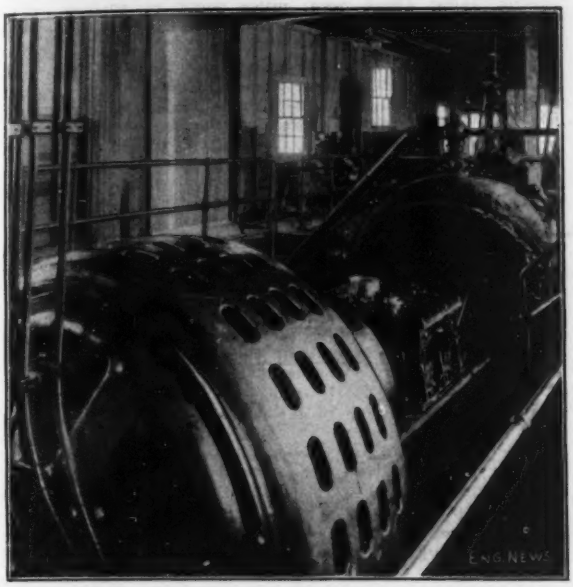

FIG. 2. MAIN CENTRIFUGAL PUMP AND MOTOR OF THE ELECTRIC SUCTION DREDGE

duction motor. There is one drum each for the two swinging cables, two quarter cables, two spud cables and a stern cable. The drum controllers for cutter and hoist motors are installed in the lever room, as are also the air-controlled valves for actuating the friction clutches and brakes for the hoisting equipment.

The compressed air for the air-control is furnished by a direct-connected compressor of 50 cu.ft. per min. capacity and an 8x8-in. compressor belted to a 24-hp. squirrel-cage induction motor. Two 35-hp. two-stage service pumps furnish water for the main pump priming ejector, main pump stuffing-box water seal, trunnion packing and bilge siphons. Only one service pump is required at a time, leaving the other for reserve. Two 10-hp. vertical bilge pumps, one at each end of the deckhouse, take care of any excess bilge water. A 30-kva. 2200-110-volt transformer furnishes current for incandescent lighting and for a 2-kw. a.c.-d.c. motor-generator searchlight set. On the top of the lever room is installed an 18-in. searchlight projector for use when night work is being done on the pontoon line or on shore.

The 20-in. discharge pipe is carried by steel pontoons of the type used generally on the Mississippi River by the U. S. Government engineers (Fig. 3). The discharge pipe is in 50-ft. lengths on the pontoons and 16-ft. lengths on shore. Connections between the pontoon pipe sections are made by means of heavy rubber sleeves fitting over the ends of the pipe. The shore lengths are adjusted by simply telescoping, no sleeves being required. At the shore end a terminal pontoon is used for the purpose of raising the end of the pontoon line to connect with the shore line. This is done by block and tackle and an A-frame, which supports the shore end of the pipe. For the pontoon line, 600 ft. to 1000 ft. of pipe are provided, and 2500 ft. for the shore line.

The electrical energy is purchased from the East St. Louis & Suburban Ry. Service is measured at the power house, the construction company bearing the losses of transmission line and transformers. The rate is based on the maximum 15-min. demand during the month, plus a unit charge per kw.-hr. consumed. The maximum demand so far is about 1050 kw. A transmission line five miles long had to be built from the power house to the dredge, about two miles of which is built along the river and within 500 ft. of the wet line at zero stage. Near one end of the river line, on poles 20 ft. from the ground, is a 13200-volt electrolytic lightning arrester.

On a housed-over barge (Fig. 4) moored to the shore are three 400 kv.a., 13,200-2200-volt electric transformers. Connecting these transformers to the transmission line is a three-conductor 13,200-volt steel-armored submarine cable 600 ft. long. On the deck of the barge, outside of the transformer house, is a 10-ft. cable reel. The end of the cable is taken into the transformer house through the hollow axle of the reel and is connected to a 13,200-volt automatic oil switch. This switch is mounted on a single panel switchboard which carries an ammeter and a voltmeter. It carries also an automatic time-limit overload relay which will automatically open the transformer oil switch in case of excessive overload. The line end of the cable terminates in a weatherproof pot-head which is clamped to a transmission line pole. Between this and the line are three choke coils which, together with the lightning arrester, constitute an effective protection against lightning. Though the company has been operating during a part of two summers, it has not suffered any injury from electrical storms.

The attachment of the cable and choke coils to the transmission line is arranged with clamps so that they may be quickly and easily removed from the line when it is desired to change the location of the dredge and

FIG. 3. DISCHARGE PIPE LINE AND REAR VIEW OF ELECTRIC SUCTION DREDGE

FIG. 4. TRANSFORMER BARGE OF TRANSMISSION LINE FOR ELECTRIC SUCTION DREDGE

pontoon line. From the low-tension side of the transformers the current is led through three 2200-volt disconnecting switches, at the lower end of which is connected a 2200-volt three-conductor submarine cable similar to the 13,200-volt cable. This cable is in one 1200-ft. length and is laid along the discharge pipe on the pontoon line. The excess cable is carried on a reel of 10-

ft. diameter, mounted on the stern of the dredge. The cable terminates in three copper rings built inside the deckhouse of the dredge, on the end of the hollow axle through which the cable leads.

Three brushes take the current from the rings, thus allowing the cable to be shortened or lengthened at will without interfering with the operation of the dredge. A five-panel switchboard is built near the back end of the deckhouse and controls all the energy used on the dredge. All motors are equipped with time-limit or instantaneous-overload protective devices, and in addition to this the main motor is equipped with a low-voltage release. An air-storage tank is installed of such capacity that in case the electrical energy should suddenly fail for any cause, there would be sufficient air to allow for setting the brakes and dogs on the swinging and hoisting equipment in order to secure all lines.

The dredge was built at the Wiggins Ferry company's sectional docks, St. Louis. The building began Apr. 4, 1912. The dredge started digging Aug. 10 and has been in successful operation 24 hours a day and seven days a week since that time, it not being necessary to shut down during the winter owing to the extremely mild weather in the Mississippi Valley.

At the end of March, 1913, after having pumped over a million yards of material, it was decided to discontinue the use of a cutter and try a straight-suction device, as the material pumped was fairly clean sand which runs freely. The cutter, cutter-shaft and cutter-head bearing were removed from the ladder, and a 45° elbow and suction pipe were substituted. This enabled the dredge to dig to a considerably greater depth than was possible when equipped with the cutter. The lower end of the suction pipe was belled out and bars were placed across the opening to prevent large pieces of wood or stone from entering. With the new device the cost of maintenance has been reduced considerably, the hours of running time have been increased, and the yardage per hour has been increased.

Compared with steam dredging, the electrically operated dredge is said to have the advantages of lower first cost, requiring less space and consequently a smaller dredge. The apparatus is simpler and more easily handled, and the likelihood of shutdowns or interruptions is less. With the muddy water of the Mississippi River, steam dredges must shut down at least once a week to clean boilers or carry an extra boiler and a complete extra condensing equipment. Troubles of transferring coal to the dredge due to low water or ice conditions are eliminated by using electricity for power.

All electrical equipment for the dredge, shore lines, lightning arrester and transformer barge (with the exception of the three 400-kv.a. transformers furnished by the Moloney Electric Co., of St. Louis) was furnished by the General Electric Co.

A Severe Fire destroyed four city blocks in the Lafayette district of Jersey City, N. J., Aug. 20. Six factories, 30 tenements and a lumber yard burned, causing an estimated loss of $1,000,000. The fire was gotten under control after a hard six hours' fight. It started in the storeroom above the office of the cooperage plant of Charles Heidt & Son, 42 Fairmount Ave. The fire got a strong start because the fire-alarm boxes at Fairmount Ave. and Virginia Ave. being pulled almost simultaneously, the forces of the firemen were divided.

The Hoisting Cage Fell to the bottom of the shaft of the Mysore gold mine, near Bangalore, India, killing 50 men.

The Explosion of a 305 mm. (12 in.) Gun while under test at Pola, Austria, Aug. 21, killed three petty officers and injured four other petty officers and two civilians, according to press dispatches. Among the officers injured was Admiral Count von Wellenburg, of the Austrian Navy, who is president of the Naval Technical Committee. Both his legs were so badly shattered that immediate amputation was necessary.

A Battleship Sank a Gunboat during recent Swedish naval maneuvers. The battleship "Oden" struck and sank the gunboat "Urd," to the westward of an island belonging to Sweden, lying about eight miles south of Elsinore. The "Oden" has a displacement of 3434 tons and the "Urd" 297 tons. The crew of the latter was saved.

The Steamship "State of California" Struck a rock and sank in Gambier Bay, near Juneau, Alaska, Aug. 17, with about 75 passengers on board. Ten bodies had been recovered and 15 passengers were known to be missing on Aug. 19. The ship is a total loss.

A Butting Collision Was Averted at the Calumet River Bridge near Hammond, Ind., Aug. 21, by means of a derailing switch. At the Calumet River the Erie R.R. and the Cleveland, Cincinnati, Chicago & St. Louis Ry. use the same bridge. A train of the latter company, carrying 12 vestibule coaches filled with passengers, was crossing this bridge when the tower man saw an Erie express train bearing down upon it. He threw the derailing switch at the approach to the bridge wrecking the locomotive and derailing the cars. The engineer and the fireman jumped as the train struck the switch and escaped injury. The other train continued on its way without lessening its speed. Wise is said to have stated that the latter train had the right-of-way and he gave it to her.

A Truck Broke under a day coach, derailing that car and three Pullmans following of the Pennsylvania R.R. eastern express from Chicago to New York over the Pittsburgh, Fort Wayne & Chicago R.R., near Leetonia, Ohio, Aug. 22. Five persons, all in the Pullmans, were injured and brought to Pittsburgh.

The Missouri Good Roads Movement, which followed the proclamation of the governor (commented upon editorially in "Eng. News," Aug. 14) took place on Aug. 20-21. According to press dispatches 250,000 men turned out. It is reported that the governor of Kansas, who aided in the work, has announced his intention to issue a proclamation for two good-road days in his state.

The Discharge of Passaic Valley Sewage into New York Bay at Robins Reef will not be enjoined by a New Jersey court, notwithstanding the efforts of the Merchants' Association of New York. Application to that end was made some weeks ago, on the ground that, since the sewage must be treated anyway, the effluent might better be discharged into Newark Bay, thus saving the cost of the proposed tunnel underneath the bay, the Bayonne peninsula and the outlet works into New York Bay. Vice-Chancellor Backes, of New Jersey, held that the discretion of the Passaic Valley Sewerage Commission governed in this case.

The Largest Dry Dock in the World is proposed for Boston, Mass., harbor, according to reports of an agreement reached on Aug. 21 between the Directors of the Port and the representatives of the International Mercantile Marine Co. and the Hamburg-American Line. By this agreement the steamship companies will pay the city $50,000 a year for the next 20 years to guarantee the construction of a dry dock at South Boston capable of taking the largest ocean steamers. This dry dock was proposed some time ago and was reported in "Engineering News," Jan. 2, 1913, p. 44. Since then the dimensions have been changed slightly, so that now it is proposed to have a basin 1200 ft. long, 150 ft. wide and 45 ft. deep at high tide. The present largest dry dock in the world is the Gladstone Dock at Liverpool, Eng., which is 1020 ft. long, 120 ft. wide and 46 ft. deep at high tide.

Wireless Communication Between the United States and South America will be established in the near future, according to a statement made Aug. 22 by William Marconi. In an address at the annual meeting of the Marconi company in London he announced that a 50-year concession had been granted by the Brazilian government, which would enable the company to inaugurate a wireless telegraph service between Brazil and New York and elsewhere in the United States. Mr. Marconi further stated that this would mark the beginning of a network of communications between Europe, the United States, and South America.

The Manufacture of Automobiles is to be abandoned by the American Locomotive Co. This company is one of the three largest makers of automobiles in the United States. The decision was reached by the board of directors on Aug. 13, but a statement of the decision, which follows, was not given out until Aug. 21:

The directors of this company at a meeting held on Aug. 13, decided to discontinue the manufacture and sale of automobiles and motor trucks. The company takes this opportunity to assure all owners of "Alco" vehicles that arrangements will be made to furnish them with repair parts for a period not less than five years to come; further, that it will fulfill in every respect its obligations, given under guarantee to customers.

The "Alco" has been one of the highest-priced cars produced in this country, selling for $6000. Of late years the company has been pushing the manufacture and sale of heavy trucks, as more profitable. In the past year, 85% of its business was in trucks and 15% in pleasure cars. The company, a few months ago, secured an order calling for the delivery of 80 large trucks for the mail-carrying service in this city. These cars are now in operation.

Important Additions to Steamship Service have been announced by both the Hamburg-American Line and the North German Lloyd S.S. Co. A passenger war between these two companies, due to disagreement over the apportionment of territory, has been imminent for some time. The North German Lloyd S.S. Co. announced on Aug. 19 that it intended to establish regular passenger and freight services, with sailings every three weeks, between Bremen and Boston and Bremen and New Orleans. The first ship from Bremen to Boston is scheduled to enter the latter port Sept. 3. The Hamburg-American Line already has service to Boston and New Orleans.

In retaliation to this plan, the Hamburg-American Line has given out that it will establish freight and steerage service between Bremen and Baltimore, the latter city being, by the pooling agreement reached several years ago, the territory of the North German Lloyd S.S. Co. The latter company, secure in its monoply of the port, avoided it in favor of Philadelphia. The Hamburg-American Line has assigned to this service the steamers "Bulgaria," 11,440 tons; the "Bosnia," 9683 tons, and the "Arcadia," 5445 tons. The tentative schedule calls for sailings every two weeks—the first ship on the schedule will be the "Arcadia," leaving Hamburg Sept. 16 and Baltimore on Oct. 8. This line also contemplates a direct service between Hamburg and Galveston, Texas, another territory of the North German Lloyd S.S. Co.

A Proposed New City Charter, which included the City Manager Plan, was rejected at Youngstown, Ohio, on July 22, according to reports, by 5596 to 2973.

The Keokuk Hydro-Electric Power Plant, generating power from the storage behind the dam across the Mississippi River at Keokuk, Iowa, was dedicated, with appropriate ceremonies, and put into initial operation on Aug. 26.

Plans for Sewage Treatment Works for Dallas, Tex., are being prepared by James H. Fuertes, of New York City. Surveys for the work have been made and construction will be carried out under the direction of J. M. Preston, city engineer, and James H. Fuertes, consulting engineer. An intercepting sewer parallel with the river will convey the sewage to a pumping station from which the sewage will be pumped about three miles to treatment works. It is expected that Imhoff tanks, sprinkling filters and secondary settling tanks will be constructed. Bonds to the amount of $550,000 were voted for this work two years ago.

The Railway-Terminal Problem of Chicago is to be investigated under the direction of a public commission composed of leading engineers and business men. This is to be undertaken for the reason that the council committee on railway terminals, which was forced by public agitation to order such an investigation made under its direction, is considered to be prejudiced in favor of the plans submitted by the Pennsylvania R.R. The committee has openly advocated these plans, and has predicted that they will be approved by the city authorities, thus indicating that it will give slight consideration to other plans or to the general problem of improvement of the railway-terminal system of the city. A review of the several projects submitted was given in our issue of July 31, 1913.

The new organization is termed the Citizen's Terminal Plan Committee, and among the engineers who are members are John W. Alvord and Samuel Dauchy. As a preliminary step, Bion J. Arnold has been retained to study and report upon the several projects noted above, to review the report which is to be made to the council committee on terminals by John F. Wallace, and to outline the requirements of an adequate and comprehensive scheme of railway-terminal facilities.

Many Small Bridges Collapsed during the past week according to clippings reaching us from various parts of the country. We give below the gist of press dispatches, with the primary cause of the bridge failure emphasized, leaving the obvious moral in each case to the reader.

Chester, Penn., Aug. 12—While Rev. J. B. Chamberlain was baptizing a dozen converts Sunday night in Naaman's Creek, a wooden bridge over the creek, on which a number of spectators were stationed, broke. Two dozen men and women were thrown into the water. None were seriously injured.

Boonville, N. Y., Aug. 12—The canal bridge collapsed yesterday afternoon, the south end of the bridge sinking into the water, carrying with it a touring car. A heavy truck carrying crushed stone for state-road work had just passed over the bridge when the auto came upon the structure. A loose board flew up and struck the front of the car, stalling the engine. After starting again the auto traveled only a few feet when the south side of the structure began to sink and gradually settled into the water. There was appropriated by the legislature two years ago $15,000 to rebuild or repair the bridge, but nothing has ever come of it.

Brookhaven, Miss., Aug. 13—The new steel bridge just completed by the County of Thayer, which was opened for traffic only yesterday, collapsed this morning at eight o'clock, three spans of 17½ ft. each, falling into the creek bottom below. There was no one on the structure at the time, but many autos had crossed, some of them weighing as much as two or three tons. The bridge was of steel with a reinforced-concrete slab floor. The steel was purchased by the county and the work of erection was done by convict labor under the direction of the Board of Supervisors. It is stated that the wreckage shows a defective piece of metal, the crack in which was covered up with paint. This gave way under the heavy load of concrete. For the last two years the Board of Supervisors has adopted the policy of erecting the steel bridges with convict labor and has effected quite a saving on several.

Port Dover, Ont., Aug. 13—A dispute involving 5c. between a picnicker and a gatekeeper at Orchard Beach here this morning caused the congestion of traffic which smashed a bridge, threw three score people a depth of 15 ft. and caused injury to a score of people. Five hundred people were proceeding quietly into Orchard Beach over an old bridge when a woman took exception to the demand of a gatekeeper for the usual charge of 5c. for her small daughter. A more or less lengthy dispute ensued and the visitors became congested on the bridge with the result noted. An examination of the bridge timbers showed them to be in bad condition, as the bridge was an old one and has often been criticised as unsafe. Only a day or two ago the hotel people who owned it took steps to mend it, but their work was not effective.

Youngstown, Ohio, Aug. 16—Forty people were thrown into 8 ft. of water in the Mahoning River here today when the Division St. foot-bridge over which a church procession was marching gave way. The bridge was washed away at the time of the recent floods and its place had been taken by a narrow plank structure.

The Single-Phase Electric-Traction System to be installed on a mountain section of the Norfolk & Western Ry., between Bluefields and Vivian, W. Va., was briefly described in "Engineering News," June 12 and Aug. 17, 1913, by announcements of the consulting engineers, Geo. Gibbs and E. R. Hill, of New York City. It was then stated that induction motors would be used but details were not given as to whether a standard polyphase type would be used with external phase transformers or some modified motor design intended to secure improved starting torque. It is now reported that the former course is to be followed, there being on the locomotive a rotary transformer connected to the main motor circuit and giving voltages a quarter cycle displaced in phase from those of the main circuit. The resulting two-phase energy supply will be used with motors of a design found satisfactory in rolling mill, hoist and similar heavy services. By special windings either four or eight poles can be produced, giving locomotive running speeds of 14 and 28 mi. per hr. For switching, two motors are to be cascade connected (secondary of the first connected to the primary of the second) giving a speed of 7 mi. per hr. Intermediate starting points are secured with resistance connected in the motor secondary circuits, the rotors being a wound-type with collector rings.

When these motors are forced to run above synchronous speed, by the momentum of trains on down grades, they act as generators feeding back energy to the supply lines and braking the train. On account of the steep grades and heavy trains of this division this consideration has been found important both on account of energy saved and the decreased wheel and brake-shoe wear involved.

It is reported also that choice of induction motors was advisable on account of cost and space limitations under the peculiar local conditions. It appears that, considering the locomotives alone, the road is to be essentially a polyphase system but, considering only the fixed equipment, it will be a single-phase line; it probably will be generally spoken of as the latter.

The Orange-Montclair-East Orange Joint Sewage-Disposal scheme, which has been under consideration for some time past, was officially adopted by the Town Council of Montclair by the passage of an ordinance on Aug. 11. The scheme involves several branch sewers and a main outlet to disposal works on a site which has already been purchased near the Third River in Belleville, and also an effluent main extending from the disposal works to the Passaic River. The disposal plant would include coarse screens, two grit chambers 55 ft. in length, each in three compartments about 4 ft. in width;

16 Imhoff settling tanks about 30 ft. wide by 82 ft. long and some 26½ ft. in depth from the water line to the bottom of the sludge chamber, with sludge compartments having a combined total capacity for at least five months' storage of sludge; eight dosing tanks; eight sprinkling filters, each about 137x119 ft in plan, having 6 ft. of clean broken stone not larger than 2½ in. in size; and sludge-drying beds having an area of about 45,000 sq.ft., composed of layers of graded gravel or broken stone with an average thickness of about 12 in., above which there would be placed a thin layer of coarse sand. The effluent will be discharged into the Passaic River in the Town of Nutley. Plans for the plant were made some time ago by Hering & Gregory, of New York City (see "Eng. News," May 9, 1912, and the plans were later approved by the State Board of Health. The cost of constructing the sewage-treatment works and the joint outlet sewer to them is to be divided in proportion to the capacity allotted to each municipality, as follows: Orange, 31.6%; Montclair, 32%; East Orange, 36.4%. Operating and repair charges are to be apportioned between the several municipalities in accordance with the volume of sewage contributed. After the official adoption of the plan by Orange and East Orange, its execution will be carried out under the direction of what is termed the "joint meeting," the members of which will be representatives from the three municipalities (all in accordance with Chap. 112, N. J. Laws of 1910).

The Bypass Channel on the Chicago River at the Jackson Boulevard bridge is now being partially destroyed in order to widen the open channel of the river. The river is very narrow at this point, having been encroached upon by land utilized for railway yards so that the west arm of the Adams St. and Jackson Boulevard swing bridges are across these yards. In 1897, the approaching completion of the Chicago Drainage Canal made it necessary to provide increased flow capacity at this point (and some other points) on the river, but the railways refused to allow the made-land to be excavated to widen the channel, and in order to avoid litigation the Chief Engineer for the Sanitary District (Isham Randolph) proposed a bypass or covered channel passing under the railway yards and on the west side of the pivot pier of each of these bridges.

The Jackson Boulevard bypass is 800 ft. long and 50 ft. wide. The side walls are of concrete, and carry heavy 48-in. plate girders spaced 10 and 15 ft. apart. Between the girders are concrete jack arches with a crown thickness of 12 to 15 in. The top of the concrete is flat and is below the level of the upper chords of the girders, but is increased in depth so as to completely encase both the upper and lower chords. Over this roof was earth filling for the tracks and team driveways. The construction of this bypass and the similar one 300 ft. long at the Adams St. bridge was described and illustrated in our issue of Feb. 16, 1899.

Negotiations in later years resulted in the railway relinquishing a part of the made land, and the Adams St. subway was torn out some time ago. The policy of widening the river for navigation and replacing swing bridges with bascule bridges calls for the removal of that part of the other bypass outside of the railway property, in connection with the reconstruction of the Jackson Boulevard bridge. For this work, the tracks and roadways were first torn up, and the shallow fill removed to expose the concrete. A floating pile-driver was moored at the head of the work and the pile-driver proper transferred to rollers on timber runways. The hammer soon broke through the arches, but numerous blows were required to break the ridge of concrete over the girders. When the concrete is all broken away the girders will be removed by derricks. Then the river wall of the channel will be destroyed, the débris being removed by dredging. The girders as exposed show the original red-lead paint very little disturbed, although in many places covered with a form of cement. A river-front wall along the new property line will be built by the railways.

The Binghamton Clothing Co. Fire, which took place at Binghamton, N. Y., July 22, involving the death of 33 women and girls and 2 men, has been described and discussed in a special report by H. W. Forster, Chairman, Committee on Safety to Life, National Fire Protection Association, 87 Milk St., Boston, Mass. (The report may be purchased for 10c. per copy.) According to the report the fire teaches the following lesson:

That the ordinary brick building with wooden floors is highly combustible and unfavorable conditions may make of it a fire flue or furnace; that combustible contents may furnish disastrous fires even in fire-resistive buildings, as was proven by the Triangle Waist Co. fire of Mar. 25, 1911; that 20 years in one factory building without a serious fire may lull the owner into an unwarranted feeling of security, and prompting to improve conditions must come from without; that in this building there was but one stairway to the third and fourth stories, and this stairway was not inclosed by a fire-resistive partition. Ordinary open stairs permit the rapid upward spread of fire and are soon made impassable. (Note) It is recognized that inclosing wooden stairs by fire-resistive partitions may not afford absolute protection. The stairs themselves can burn, as in this case, or the doors may be open, and smoke and flames thereby get into the shaft. The so called Philadelphia smoke-proof fire tower is the only safe type of vertical exit. A combination of inclosed stairways for ordinary use and of one or more fire towers for emergency exit purposes makes the ideal arrangement.

Reliance for emergency exit from this building was placed upon one outside iron fire escape, a type of exit which, though allowed by law, is condemned by the experience of many fires. (Note) To have any value at all in a building where many persons are employed, a fire escape must be ample as regards strength, have easy stairs and reasonably wide walkways, permanent stairs to the ground, or swinging counterbalanced stairs from the second floor to the ground, and either be located against a blank brick wall or have contiguous window openings protected by wired glass.

Fire drills alone do not insure safety of life in a fire trap. (Note) The one open stairway and iron fire escape which constituted the means of exit from the fourth floor where eight persons were employed were together incapable of accommodating more than about forty persons, and this overcrowding or excessive occupancy on the top floor placed the employees there in constant danger.

That fires when promptly discovered can frequently not be handled by the local people because of lack of equipment, lack of knowledge of how to handle equipment, or because people lose their heads. (Note) Long experience and continually repeated demonstration prove that the automatic sprinkler, where properly installed and in operating condition, is always ready, operates quickly, and either extinguishes fire or holds it in check, and is the most reliable means of safeguarding lives in the majority of existing manufacturing buildings. Inclosing stairs, building fire towers, properly constructing fire escapes, etc., are all means intended to permit of escape after the fire gets under way. The automatic sprinkler almost invariably prevents the fire from assuming serious proportions.

Three articles recently published in "Engineering News" have as their common object the reduction of fire hazard. The articles referred to are "Life Hazard in Crowded Buildings Due to Inadequate Exits," "Standard Specifications for Water-Works' Hydrants and Valves" and "Standardization of Hose and Hydrant Connections," published July 31, Aug. 7 and Aug. 14, 1913, respectively.

PERSONALS

Mr. Joseph Firth, Assoc. M. Am. Soc. C. E., has resigned as City Engineer of Charlotte, N. C.

Mr. Ward D. Carpenter has resigned as Engineer for the Cameron County Drainage District No. 3, in Texas.

Mr. W. Osborne has been appointed Engineer of Maintenance-of-way of the Atlantic Southern R.R., Atlantic, Iowa.

Mr. C. A. Cottrell, Trainmaster of the Canadian Pacific Ry., at Vancouver, B. C., has been promoted to be Superintendent, succeeding Mr. George A. Graham.

Mr. H. B. Adams, recently general foreman of the Gulf, Colorado & Santa Fé Ry., at Cleburne, Tex., has been promoted to be Master Mechanic at Silsbee, Tex.

Mr. A. Kauffman, recently Assistant to the President of the Link-Belt Co., Chicago, Ill., has been appointed General Manager of the Philadelphia Works, at Nicetown, Penn.

Mr. J. E. Hood has been appointed Supervisor of Bridges and Buildings of the Northern Pacific Ry., at Duluth, Minn., succeeding Mr. R. E. McFarlane, on leave of absence.

Mr. E. P. Goodwin, General Inspector of Transportation of the Chesapeake & Ohio Ry., has been promoted to be General Superintendent of Transportation, with headquarters at Richmond, Va.

Mr. C. C. Walker, recently General Superintendent of Transportation of the Chesapeake & Ohio Ry., has been promoted to be Assistant to the General Manager, with offices at Richmond, Va.

Mr. William F. Woerner has been appointed the fifth member of the new Public Service Commission of Missouri. The appointment of the other four members was noted in our issue of May 1, 1913.

Mr. H. C. Van Buskirk has been appointed Mechanical Superintendent of the First District of the Chicago, Rock Island & Pacific Ry., at Des Moines, Iowa, succeeding Mr. J. B. Kilpatrick, resigned.

Mr. F. C. Stuart has resigned as Signal Engineer of the Texas & New Orleans R.R., the Galveston, Harrisburg & San Antonio Ry. and Morgan's Louisiana & Texas R.R. & Steamship Co., Houston, Tex.

Mr. R. P. Stevens, who recently resigned as President of the Lehigh Valley Transit Co., has been elected President of the Mahoning & Shenango Ry. & Light Co., Youngstown, Ohio, succeeding Mr. N. M. Crawford, resigned.

Mr. M. A. Neville, Superintendent of the Peoria & Eastern division of the Cleveland, Cincinnati, Chicago & St. Louis Ry., has been appointed General Superintendent of the Cincinnati Northern R.R., with office at Van Wert, Ohio.

Mr. E. I. Ford, Superintendent of Terminals at Newport News, Va., of the Chesapeake & Ohio Ry., has been transferred to be Superintendent of the Richmond division, at Richmond, Va., succeeding Mr. H. H. Norris, transferred.

Mr. George D. Marshall has resigned as Highway Engineer of Bexar County, Texas. After a vacation of several weeks he will return to his former position of Superintendent of Road Construction under the U. S. Office of Public Roads.

Mr. J. F. Bowden, formerly Master Mechanic of the Baltimore & Ohio R.R., at Newark, Ohio, has been promoted to be District Superintendent of Motive Power, at Wheeling, W. Va., succeeding Mr. G. A. Schmoll, transferred to Pittsburgh, Penn.

Mr. Hugh Montgomery, formerly Superintendent of Motive Power and Equipment of the Bangor & Aroostook R.R., Milo Junction, Maine, has been appointed Superintendent of Motive Power and Rolling Stock of the Rutland R.R., at Rutland, Vt., succeeding Mr. F. C. Cleaver, resigned.

Mr. J. E. Taussig, formerly Division Superintendent of the Galveston, Harrisburg & San Antonio Ry., at San Antonio, Tex., has been appointed Special Representative on the staff of Mr. E. F. Kearney, First Vice-President of the Texas & Pacific Ry., with headquarters at New Orleans, La.

Mr. J. W. Knapp, recently General Superintendent of the Virginia division of the Chesapeake & Ohio Ry., at Richmond, Va., has been appointed Superintendent of Terminals at Newport News, Va. He is succeeded at Richmond by Mr. J. Paul Stevens, recently General Superintendent at Cincinnati, Ohio.

Mr. Charles H. Moyer, for the past 20 years New York Manager of the Geo. V. Cresson Co., Philadelphia, Penn., and of its successor the Cresson-Morris Co., has resigned to engage in business as a special engineering representative and manufacturers' agent with office at 90 West St., New York City.

Mr. W. R. Hudson, recently Superintendent of the Chesapeake & Ohio Ry. of Indiana, at Peru, Ind., has been promoted to be General Superintendent of the Kentucky division, at Cincinnati, Ohio, succeeding Mr. J. Paul Stevens, transferred. He is succeeded at Peru by Mr. H. H. Norris, transferred from the Richmond (Va.) division.

Mr. L. W. Barnette, for several years Secretary and Sales Manager of the Franklin Steel Co., Franklin, Penn., has resigned to accept a position with the Blaw Steel Construction Co., of Pittsburgh, Penn., with headquarters at 165 Broadway, New York City. He will have charge of Eastern sales department for transmission towers, steel poles and structural steel.

Mr. Fred C. MacMillan, Vice-President of the St. Paul & Kansas City Short Line R.R., at Des Moines, Iowa, has resigned to become Vice-President of the San Antonio, Uvalde & Gulf R.R., at San Antonio, Tex. The office of Vice-President of the St. Paul & Kansas City Short Line R.R. has been abolished and the duties of that office will be assumed by Mr. W. M. Whitenton, General Manager.

Mr. Henry Floy, M. Am. Soc. C. E., Consulting Engineer, New York City, has been appointed Appraiser of the assets of the John F. Stevens Construction Co., 55 Wall St., New York City, which was recently forced into the hands of a receiver, as noted in our Construction News columns of Aug. 14. The company has uncompleted contracts for sections of the New York City subways, hydro-electric developments and other large construction work.

Lieut.-Col. George A. Zinn, M. Am. Soc. C. E., Corps of Engineers, U. S. A., recently in charge of harbor improvements at Chicago, Ill., and other Lake ports, and of the Illinois River improvement, has been transferred to Philadelphia, Penn., to take charge of the extensive improvements in the Delaware River and the port of Philadelphia. Lieut.-Col. Zinn succeeds Lieut.-Col. Joseph E. Kuhn, who has been assigned to special duty at Washington, D. C.

Messrs. Oliver H. Hughes, of Hillsboro, Ohio; Judge William L. Dechant, of Middletown; and Edward W. Doty, of Cleveland, have been appointed members of the Ohio Public Utilities Commission, which replaces old Public Service Commission under a law passed during the recent session of the legislature. Mr. Hughes was a member of the former Public Service Commission; Judge Dechant is a lawyer and banker, and Mr. Doty is a lawyer and former member of the legislature.

Prof. Edward D. Rich, M. Am. Soc. C. E., Assistant Professor of civil engineering at the University of Michigan has been appointed State Sanitary Engineer of Michigan. Prof. Rich graduated from Rensselaer Polytechnic Institute in 1895. His early experience in engineering was received in the city engineer's office at Syracuse, N. Y., and as leveler and Assistant Engineer on the New York State Canals. For five years he was Assistant City Engineer of Utica, N. Y., and for four years City Engineer of Summit, N. J. He has been a member of the faculty of the University of Michigan since 1908.

Prof. Lewis E. Young, for the past six years Director of the Missouri School of Mines, has resigned in order to take up graduate work in the department of economics at the University of Illinois. He will also devote a part of his time to teaching in the department of mining engineering. Prof. Young graduated from the Pennsylvania State College in 1900, and has since studied at the Michigan College of Mines and at the University of Chicago. He was formerly Professor of mining at the Colorado School of Mines. For some years Prof. Young has made a special study of the economics of mining, taxation and political economy, and he will carry on advanced work in these subjects at the University of Illinois.

Mr. Elmer A. Holbrook, M. Am. Inst. M. E., Assistant Professor of mining and metallurgy at the Nova Scotia Technical College, Halifax, N. S., has been appointed Assistant Professor of mining engineering at the University of Illinois. He will have charge of the recently equipped coal-washing and ore-dressing laboratory and the course in mine design. Prof. Holbrook graduated from the Massachusetts Institute of Technology in 1904, and he has since spent five years in active mine work in the United States, Canada and Mexico. For the past three years he has been Assistant Professor at the Nova Scotia Technical College, where he had charge of the building of the mining laboratory. He has also carried out some extensive tests on the washing of Nova Scotia coals.

Mr. George Bradshaw has been appointed Safety Engineer of the Grand Trunk Ry. This is a new office, the duties of which will include the inspection of terminals, shops, etc., conferences with various operating officials on matters pertaining to the safety of both the traveling public and the railway employees. The Safety Engineer, assisted by local officers, will hold meetings at all important centers on the Grand Trunk System, at which practical instruction bearing on safety will be given to employees of all departments. Lantern slides of safe and unsafe conditions will be shown and explained. Committees composed of officers and employees will be organized under the direction of the Safety Engineer on all divisions, whose duty it will be to discover and correct unsafe conditions and practices.

Mr. William B. Fuller, M. Am. Soc. C. E., of Johnson & Fuller, Civil and Sanitary Engineers, New York City, returned to New York, Aug. 25, from La Boquilla, Mex., after having spent two years in charge of the construction of a large dam there for a hydro-electric development promoted by Canadian interests. Mr. Fuller has resigned his position as Chief Engineer of Construction of the dam, having had quite enough of Mexico under the present conditions. The work has been carried on under exceptional difficulties, as the readers of Mr. Fuller's article in "Engineering News" of May 23, 1912, and of subsequent personal and news notes in our issues since then, must be fully aware. La Boquilla has been the center of a revolutionary district from which all supplies were frequently cut off for months at a time. The construction camp has been occupied from time to time by many different political forces, alternately Federals, Rebels, Constitutionalists—and just plain Bandits—all of whom, Mr. Fuller says, made pointed demands for money, and were very careless in the use of their rifles when collecting contributions. Finally, on being threatened with instant execution by an approaching Federal force, Mr. Fuller waited until this force was within four miles of camp and then decided there were healthier places for him than La Boquilla. Accordingly he made his escape on horseback to a ranch 50 miles away, from which place he was able to continue his flight more comfortably by automobile.

OBITUARY

N. L. Barnum of Marshall, Mo., a young civil engineer in charge of doubling-tracking work on the St. Louis Southwestern Ry., near Dexter, Mo., was struck and killed by lightning, on Aug. 19. He was 26 years old.

Lucien Hougnet Blythe, a civil engineer employed by the American Bridge Co., died at his home in Brooklyn, N. Y., Aug. 17. He was born in Philadelphia, Penn., June 13, 1880,

and was educated at the University of Pennsylvania. He is survived by a widow and one son.

James Richard Bell, M. Inst. C. E., famous as the builder of several notable bridges in India, died Aug. 8, at his home, Hazeldene, Ightham, Kent. He was born in 1841 and received his education at Aberdeen in Scotland. He had been a member of the Institution of Civil Engineers since 1879; he was elected a member of the American Society of Civil Engineers in 1896.

Florentin Pelletier, a former civil engineer and architect, died at his home in Plainfield, N. J., Aug. 22, aged 88 years. He was at one time employed in the government lighthouse service on lighthouse construction in Boston harbor. He was also engaged in engineering work on the construction of the Ogdensburg & Lake Champlain R.R. He retired from active work some 12 years ago.

Fred Gordon Plummer, Geographer of the United States Forest Service, was found dead in his home in Washington, D. C., Aug. 27. He was living alone during the summer months and is supposed to have died from heart disease Aug. 15. He was born in New York City Oct. 4, 1864. His earliest engineering experience was as rodman and levelman on special work in connection with levee construction on the lower Mississippi River in 1883. The following year he was engaged in survey work in Tacoma, Wash. Later returned East to study in New York and Boston, after which he went back to Washington State as Assistant Curator of the Agassiz Museum at Olympia. After two years as Instructor at Washington College at Tacoma he again took up engineering and survey work as Chief Engineer for the State University land and building commission, the Tacoma water-works commission and as County Engineer. Later he was employed in special work by the Northern Pacific Ry. From 1898 to 1903 Mr. Plummer was Special Field Assistant of the U. S. Geological Survey detailed to the examination of the national forest lands. In 1903 he passed the civil-service examinations and was appointed Irrigation Engineer and Hydrographer in the Geological Survey. In July, 1905, he was transferred to be Civil Engineer of the Forest Service. Mr. Plummer was the author of many reports and publications on geology, geography and forestry, and is credited with having written the first technical description of the forest conditions in many of the national forests in Washington, Oregon, Arizona, New Mexico and California. His chief work for the Forest Service is said to have been his compilation of statistical and map data. He was a member of the Washington Academy of Science, the Society of American Foresters, and other scientific societies. He is survived by a widow and five children.

Franklin Allen Hinds, M. Am. Soc. C. E., Consulting Engineer, of Watertown, N. Y., died Aug. 23, near Boston, Mass., while on an automobile tour with his wife and brother. Mr. Hinds was born Nov. 17, 1843, on his father's farm near Watertown, N. Y., where he received his elementary education in the public schools. At 21, he went to Portland, Ore., by way of the Panama route. In Portland, he studied under various engineers for two years, then returned East where he studied for a year at Yale University.

Mr. Hinds then undertook surveys for the Carthage, Watertown & Sacketts Harbor R.R., and later he was made Chief Engineer of this railroad. Other railroad-engineering work of which he had charge were surveys for the Kingston & Pembroke R.R. and for the New York & Boston Inland R.R. For some years he was in partnership with Mr. John Moffett, and they were engaged in constructing municipal water-supplies in all parts of the United States and Canada.

In 1889, Mr. Hinds formed a partnership with Mr. E. A. Bond, Ex-Chairman of the New York Barge Canal Advisory Board. Together, they had a large consulting practice in mill construction, water-works, hydro-electric developments and general practice in engineering. This partnership was dissolved in 1896, and the business was continued by Mr. Hinds until his death. He was City Engineer of Watertown for several years, and a member of the Board of Watertown Commissioners for 33 years, and a vestryman of Trinity Church for 26 years.

Through his sterling character and his kindly interest in others, Mr. Hinds was always an inspiring example to the younger men with whom he came in contact. He loved his profession and instilled that love in others. To him, truth and honesty were fundamental, and no design was good, or engineering project sound unless it was founded on truth and designed with unswerving honesty. Mr. Hinds was married in 1867 to Miss Mary R. Thompson, who, with one brother, Oscar E. Hinds, survives him.—[Contributed by Lou B. Cleveland, Assoc. M. Am. **Soc. C. E., Civil Engineer and Contractor**, Watertown, N. Y.]

ENGINEERING SOCIETIES
COMING MEETINGS

AMERICAN BOILER MANUFACTURERS' ASSOCIATION.
Sept. 1-4. Annual convention at Cleveland, Ohio. Secy., J. D. Farasey, 37th St., Cleveland, Ohio.

INTERNATIONAL ASSOCIATION OF FIRE ENGINEERS.
Sept. 1-6. Annual convention at New York City. Secy., Jas. McFall, Roanoke, Va.

ROADMASTERS' AND MAINTENANCE OF WAY ASSOCIATION.
Sept. 8-12. Annual convention at Chicago, Ill. Secy., L. C. Ryan, Chicago and North Western Ry., Stirling, Ill.

AMERICAN ELECTROCHEMICAL SOCIETY.
Sept. 9-11. The 24th general meeting at Denver. Secy., Prof. Jos. W. Richards, Lehigh University, South Bethlehem, Penn.

NEW ENGLAND WATER-WORKS ASSOCIATION.
Sept. 10-12. Annual convention at Philadelphia, Penn. Secy., Willard Kent, Narragansett Pier, R. I.

AMERICAN PUBLIC HEALTH ASSOCIATION.
Sept. 9-13. Annual meeting at Colorado Springs, Colo. Secy., S. M. Gunn, 755 Boylston St., Boston.

MASTER CAR AND LOCOMOTIVE PAINTERS' ASSOCIATION.
Sept. 9-12. Annual convention at Ottawa, Ont. Secy., A. P. Dane, Reading, Mass.

NATIONAL PAVING BRICK MANUFACTURERS ASSOCIATION.
Sept. 17-18. Tenth annual meeting at Cleveland, Ohio. Secy., W. P. Blair, 824 Engineers' Building, Cleveland, Ohio.

INTERNATIONAL CONGRESS OF REFRIGERATION.
Sept. 17-Oct. 1. Third congress, at Chicago, Ill. Secy., J. F. Nickerson, Chicago.

ILLUMINATING ENGINEERING SOCIETY.
Sept. 22-26. Annual convention at Pittsburgh, Penn. Secy., I. D. Israel, 29 W. 39th St., New York City.

AMERICAN MINE SAFETY ASSOCIATION.
Sept. 22-24. Annual meeting at Pittsburgh, Penn. Chairman, H. M. Wilson, 40th and Butler Sts., Pittsburgh, Penn.

AMERICAN ROAD CONGRESS.
Sept. 29-Oct. 5. Annual convention at Detroit, Mich. Gen. Secy., J. C. Pennybacker.

AMERICAN SOCIETY OF MUNICIPAL IMPROVEMENTS.
Oct. 7-10. Annual convention at Wilmington, Del. Secy., Geo. H. McGovern, Chamber of Commerce, Wilmington, Del.

RAILWAY SIGNAL ASSOCIATION.
Oct. 14-16. Annual meeting at Nashville, Tenn. Secy., C. C. Rosenberg, Times Building, Bethlehem, Penn.

AMERICAN ELECTRIC RAILWAY ASSOCIATION.
Oct. 13-17. Annual convention at Atlantic City, N. J. Secy., H. C. Donecker, 29 West 39th St., New York City.

AMERICAN GAS INSTITUTE.
Oct. 15-17. Annual meeting at Richmond, Va. Secy., Geo. G. Ramsdell, 29 West 39th St., New York City.

AMERICAN MINING CONGRESS.
Oct. 20-25. Annual convention at Philadelphia, Penn. Secy., J. Calbreath, Majestic Building, Denver, Colo.

AMERICAN RAILWAY BRIDGE AND BUILDING ASSOCIATION.
Oct. 21-23. Annual convention at Montreal, Que. Secy., C. A. Lichty, 319 North Waller Ave., Chicago, Ill.

NATIONAL ASSOCIATION OF RAILWAY COMMISSIONERS.
Oct. 28. Annual Convention at Washington, D. C. Secy., Wm. H. Connolly, Interstate Commerce Commission, Washington, D. C.

Iron and Steel Institute—The autumn meeting will be held at the Palais des Académies, Brussels, on Sept. 1-4. Among the papers to be presented are the following: "The Manufacture of Armor-Piercing Projectibles"; "Commercial Production of Sound Steel Ingots"; "Present Methods of Testing, with Special Reference to the Work of the International Association for Testing Materials"; "Modern Open-Hearth Steel Furnaces"; "The Use of Oxygen in Blast Furnaces"; "Note on the Principal Deposits of Iron Ore in Chili."

International Congress of Refrigeration—The third congress will be held at Chicago from Sept. 17 to Oct. 1. The first congress was held at Paris in 1908, and the second at Vienna in 1910. The proceedings will be divided among six sections, each of which will hold a morning or afternoon meeting each day, the balance of the time being devoted to excusions and entertainments. These sections are as follows: (1) Liquid gases and units of refrigeration; (2) Refrigerating machinery and insulating materials; (3) Refrigeration for preserving foods; (4) Refrigerating apparatus in the industrial arts; (5) Refrigeration in railway and steamship transportation; (6) Legislation and administration.

These congresses are held under the direction of the International Association of Refrigeration, the President of which is Andre Lebon, of Paris. Dr. David Franklin Houston, Secretary of Agriculture, is Honorary President of the present congress; Dr. F. W. Gunsaulus, of Chicago, is President; and **J. F. Nickerson, of Chicago, is Secretary.**